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Effects of Shear Reinforcement on the Large-Deflection Behavior of Reinforced Concrete Slabs

by Stanley C. Woodson

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U.S. Army Corps of Engineers
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3909 Halls Ferry Road
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Final report

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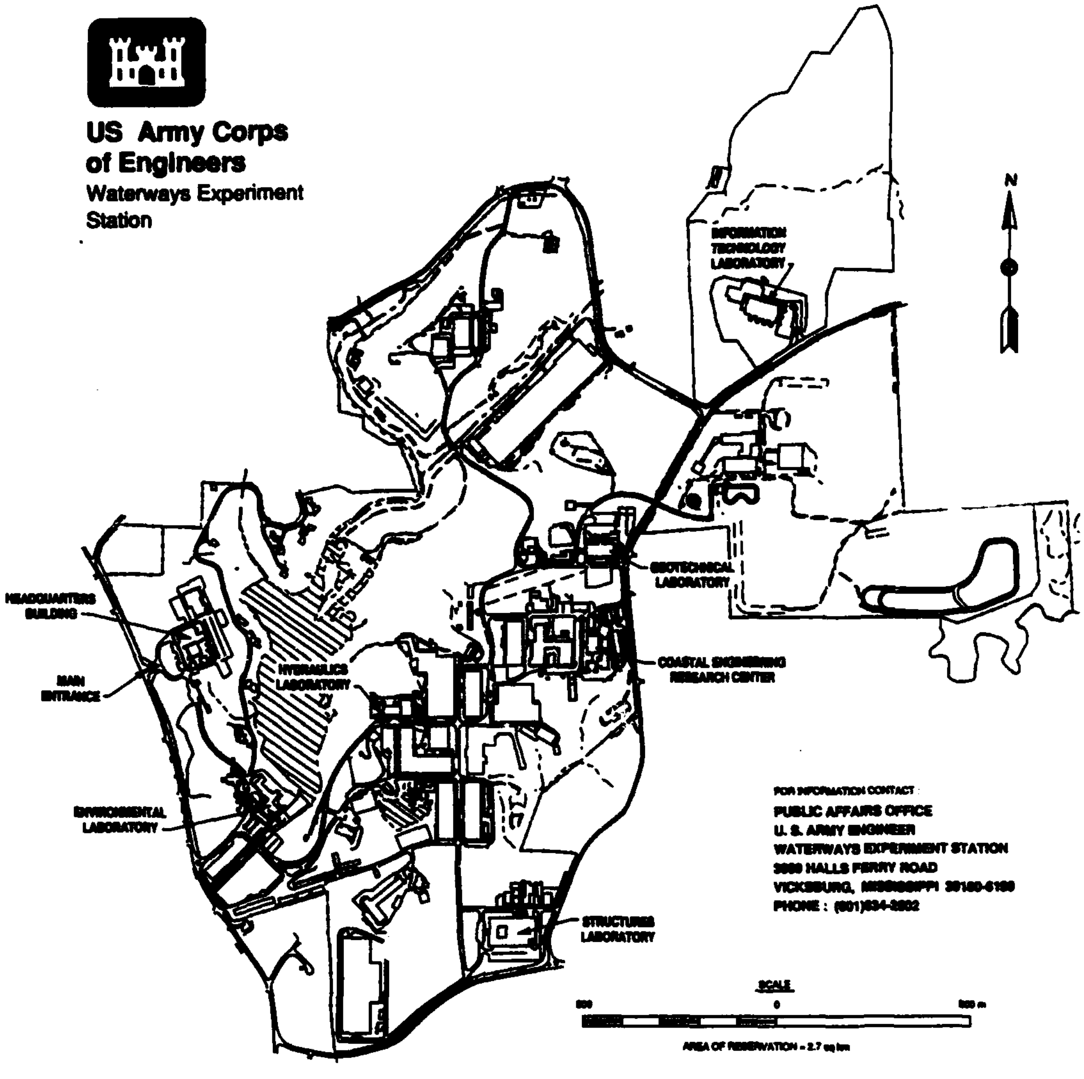

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PREFACE

This study was conducted by the U.S. Army Engineer Waterways Experiment Station (WES) under the joint sponsorshp of the Laboratory Discretionary Research and Development Program at WES and the Department of Defense Explosives Safety Board.

The work was conducted at WES in the Structures Laboratory under the supervision of Messrs. Bryant Mather, Director, James T. Ballard, Assistant Director, and Dr. Jimmy P. Balsara, Chief, Structural Mechanics Division (SMD). This report was prepared by Dr. Stanley C. Woodson, SMD, and was submitted to the University of Illinois at Urbana-Champaign in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Civil Engineering.

Dr. Robert W. Whalin was Director of WES. COL Bruce K. Howard, EN, was Commander.

CONTENTS

CHAPTER 1

INTRODUCTION

1.1 Background

Some design guides and manuals for blast-resistant reinforced concrete structures stipulate the use of shear reinforcement in roof; floor, and wall slabs irrespective of shear stress levels. In such cases, the primary purpose of shear reinforcement is not to resist shear forces, but rather to improve performance in the large-deflection region by tying the two principal reinforcement mats together. Shear reinforcement used in blast-resistant slab design usually consists of either lacing bars or single-leg stirrups (Figure 1.1). Lacing bars are reinforcing bars that extend in the direction parallel to the principal reinforcement and are bent into a diagonal pattern between mats of principal reinforcement. The lacing bars enclose the transverse reinforcing bars (often referred to as temperature steel in one-way slabs) which are placed external to the principal reinforcement for a laced slab. The cost of using lacing reinforcement is considerably greater than that of using single-leg stirrups due to the more complicated fabrication and installation procedures.

Section 4.23.1 of the Tri-Service Technical Manual (TM) 5-1300 (1) provides some discussion on construction economy. It states that construction costs are divided between labor and material costs, with labor cost accounting for as much as 70 percent of the cost of blast-resistant concrete. TM 5-1300 states that the initial design, optimized for material quantities, may need to be modified when constructability is considered. It

further states that such a modification may actually increase the total cost of materials for the structure while reducing labor-intensive activities. It is generally known that the fabrication and installation of large quantities of shear reinforcement, particularly that having a complex configuration (such as lacing bars), are labor-intensive activities.

In the design of conventional structures, the primary purpose of shear reinforcement is to prevent the formation and propagation of diagonal tension cracks. The shear reinforcement requirements for conventional structures are based on much research and data, particularly, from statically tested beams. Relatively little study has been devoted to examining the role of shear reinforcement in slabs subjected to distributed dynamic loads, especially in the large-deflection region of response. In blast-resistant design, structures are typically designed to survive only one loading and relatively large deflections are acceptable as long as catastrophic failure is prevented.

A considerable amount of recent (1970's and 80's) data from various experiments conducted on slabs indicated that the shear reinforcement design criteria that is typical of design manuals such as TM 5-1300 may be excessive. A data base, including static and dynamic tests conducted from the 1960's through the early 1990's, is presented in this thesis. The data base consists primarily of slab tests conducted to investigate parameters other than shear reinforcement details. Consequently, a thorough study of the role of shear reinforcement (stirrups and lacing) in slabs designed to resist blast loadings or undergo large deflections has never been conducted. A better understanding of the contributions

of the *shear reinforcement* will allow the designer to evaluate the benefits of using shear reinforcement and to determine which type is most desirable for the given structure. This capability will result in more efficient and effective designs as reflected by lower cost structures without the loss of blast-resistant capacity. As presented herein, a *reasonable first* step toward this goal is to perform a series of laboratory experiments that compare the effects of stirrups and lacing bars on the large-deflection behavior of one-way slabs.

1.2 Objective

The overall objective of this study is to better understand the effects of shear reinforcement details on slab behavior in order to improve the state-of-the-art in protective construction design, for both safety and cost effectiveness. This is not particularly a study of shear stresses in slabs, but rather a study of the effects of shear reinforcement on the large-deflection behavior of slabs.

Specifically, the objective is to evaluate and compare the effectiveness of stirrups and lacing bars in enhancing the ductility of one-way slabs. This must include a consideration of how shear reinforcement details interact with other physical details to affect the response of a slab. The work reported herein is directed toward the development of new guidelines for shear reinforcement requirements in blast-resistant structures.

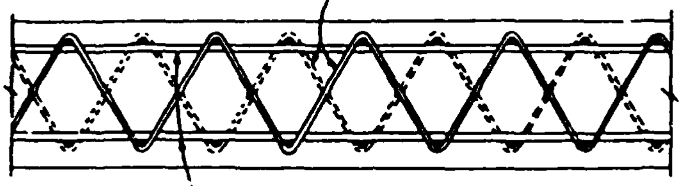
1.3 Scope

In order to understand the development of current design criteria and to document recent data, a literature survey was

conducted in search of data obtained from experiments where reinforced concrete slabs were loaded to failure or to large deflections (statically and dynamically). The available data were in the form of research papers and technical reports. Of course, different authors addressed different concepts and details; therefore, the design parameters that were presented and emphasized varied among the reports.

The known design/construction parameters and other parameters (such as ultimate resistance, secondary resistance, maximum deflection, support rotation, loading technique, and extent of damage) associated with the structural response of the slabs were tabulated and entered into a Lotus 1-2-3 file for future manipulation. Discussion of the data is presented in this paper. Also, a summary of current design criteria found in the design manuals is presented, and data are compared to the criteria.

Sixteen one-way reinforced concrete slabs were statically (slowly) loaded with water pressure in the 4-foot-diameter blast load generator located at the U.S. Army Engineer Waterways Experiment Station (WES). The design, construction, and loading of the specimens are described herein. The responses of the slabs to the uniform loading and the effects of the reinforcement details on the responses are evaluated.



a. Lacing Reinforcement

b. Single-leg Stirrup

Figure 1.1. Shear Reinforcement

CHAPTER 2

CURRENT PRACTICE

2.1 Introduction

In order to form an understanding of the intended role of shear reinforcement in structures designed to resist conventional weapons effects, summaries of selected design manuals or guidance documents are given in this chapter. The reader's familarality with current design criteria, as described below, is essential for recognizing the significance of the data presented and discussed in subsequent chapters of this thesis.

In conventional building design the primary source of design guidance for the placement of reinforcing steel in reinforced concrete structures, including shear reinforcement, is that of the American Concrete Institute's (ACI) Committee 318 (2). No such single, widely accepted criteria document exists for blast-resistant design guidance; however, the most widely used reference for protective design in the area of explosive safety (pertaining to non-nuclear accidental explosions) is TM 5-1300 (1). Other prominent guidance documents include the Army manual on protective construction for conventional weapons effects, TM 5-855-1 (3); a recent supplement to TM 5-855-1, Engineer Technical Letter (ETL) 1110-9-7 (4); and the semi-hard design criteria document published by the U.S. Air Force (5). Summaries of the guidance for shear reinforcement from each of these design documents follow.

2.2 The Tri-Service Technical Manual 5-1300

Intended primarily for explosives safety applications, the TM 5-1300 (Army designation) is the most widely used manual for

structural design to resist blast effects from conventional weapons or explosives. Its Navy designation is NAVFAC P397, and for the Air Force it is AFM 88-22. For convenience it will only be referred to as TM 5-1300 in this thesis. The manual was recently revised into six chapters. Chapter 4 of TM 5-1300 deals with reinforcement details and will be the primary portion of the manual discussed.

In Section 3-11 of the original TM 5-1300 (6) that was published in 1969, the use of lacing was required for "close-in" detonations, i.e. whenever pressures much larger than 200 psi were expected. The use of nonlaced concrete elements was allowed at lower pressures if a maximum support rotation (θ), defined simply as the arctan of the quantity given by the midspan deflection divided by one-half of the clear span length, of less than 2 degrees was predicted. These restrictions have been relaxed slightly in Chapter 4 of the current version of TM 5-1300 as follows. Considering the resistance-deflection relationship for flexural response of a reinforced concrete element, Section 4-9.1 of the current manual states that, within the range following yielding of the flexural reinforcement, the compression concrete crushes at a deflection corresponding to 2 degrees support rotation. This crushing of the compression concrete is considered to be "failure" for elements without shear reinforcement. For elements with shear reinforcement (single-leg stirrups or lacing reinforcement) which properly tie the flexural reinforcement, the crushing of the concrete results in a slight loss of capacity since the compressive force is transferred to the compression reinforcement. As the reinforcement enters into its

strain-hardening region, the resistance increases with increasing deflection. Section 4-9.1 of the manual states that single-leg stirrups will restrain the compression reinforcement for a short time into its strain hardening region until the element loses its structural integrity and failure occurs at a support rotation of 4 degrees. It further states that lacing reinforcement will restrain the flexural reinforcement, through truss action, through its entire strain-hardening region until tension failure of the principal reinforcement occurs at a support rotation of 12 degrees.

TM 5-1300 distinguishes between a "close-in" design range and a "far" design range for purposes of predicting the mode of response. In the far design range, the distribution of the applied loads is considered to be fairly uniform and deflections required to absorb the loading are comparatively small. Section 4-9.2 states that nonlaced elements are considered to be adequate to resist the far-design loads with ductile behavior within the constraints of the allowable support rotations discussed in the preceding paragraph. The design of the element to undergo deflections corresponding to support rotations between 4 and 12 degrees requires the use of laced reinforcement. An exception is when the element has sufficient lateral restraint to develop in-plane forces in the tensile membrane region of response. In this case, Section 4-9.2 states that the capacity of a nonlaced element increases with increasing deflection until the reinforcement fails in tension. A value of support rotation is not given here, but one might deduce that a support rotation of 12 degrees is intended since it is the value given in Section 4-9.1

for tension failure of the reinforcement in a laced slab. However, a value of 8 degrees is given elsewhere in the manual as a limit of support rotation for elements containing stirrups and experiencing tensile membrane behavior.

Section 4-9.3 of TM 5-1300 discusses ductile behavior in the close-in design range. Again, the maximum deflection of a laced element experiencing flexural response is given as that corresponding to 12 degrees support rotation. This section states the following:

"Single leg stirrups contribute to the integrity of a protective element in much the same way as lacing, however, the stirrups are less effective at the closer explosive separation distances. The explosive charge must be located further away from an element containing stirrups than a laced element. In addition, the maximum deflection of an element with single leg stirrups is limited to 4 degrees support rotation under flexural action or 8 degrees under tension membrane action. If the charge location permits and reduced support rotations are required, elements with single leg stirrups may prove more economical than laced elements."

Section 4-32 of TM 5-1300 states:

"... Also, the blast capacity of laced elements are greater than corresponding (same concrete thickness and quantity of reinforcement) elements with single leg stirrups. Laced elements may attain deflections corresponding to 12 degrees support rotation whereas elements with single leg stirrups are designed for a maximum rotation of 8 degrees. These nonlaced elements must develop tension membrane action in order to develop this large

support rotation. If support conditions do not permit tension membrane action, lacing reinforcement must be used to achieve large deflections."

It is stated throughout TM 5-1300 that laced elements may attain support rotations of 12 degrees whether or not they are restrained against lateral movement at the supports. The manual also implies that a nonlaced element may only achieve its maximum support rotation of 8 degrees when it is restrained against lateral movement.

In addition to being required for large-deflection behavior, lacing reinforcement is always required in slabs subjected to blast at scaled distances less than 1.0 ft/lbs$^{1/3}$. Section 4-9.4 of the TM 5-1300 indicates that lacing reinforcement is required due to the need to limit the effects of post-failure fragments resulting from flexural failure. It is stated that the size of failed sections of laced elements is fixed by the location of the yield lines, whereas the failure of a nonlaced element results in a loss of structural integrity and produces fragments in the form of concrete rubble. Section 4-22 discusses the use of single-leg stirrups in slabs at scaled distances between 1.0 and 3.0 ft/lbs$^{1/3}$, which are considered to be respectively the lower and upper bounds of the close-in range for slabs with stirrups. Support rotations in slabs with stirrups are limited to 4 degrees in the close-in design range unless support conditions exist to induce tensile membrane behavior. Another distinction given between laced and nonlaced elements is that a nonlaced element designed for "small" deflections in the close-in design range is considered to not be reusable (no multiple loadings).

2.3 Army Technical Manual 5-855-1

TM 5-855-1 (3) is intended for use by engineers involved in designing hardened facilities to resist the effects of conventional weapons. The manual includes design criteria for protection against the effects of a penetrating weapon, a contact detonation, or the blast and fragmentation from a standoff detonation.

Chapter 9 of TM 5-855-1 discusses the design of shear reinforcement. The criteria presented are based primarily on the guidance given in the 1983 edition of ACI 318 with consideration of available test data. The maximum allowable shear stress to be contributed by the concrete and the shear reinforcement is given as $11.5(f'_c)^{1/2}$ for design purposes. An upper bound to the shear capacity of members with web reinforcing is given as that corresponding to a 100 percent increase in the total shear capacity outlined by ACI 318-83 and consisting of contributions from the concrete and shear reinforcement. An important statement concerning shear reinforcement in one-way slabs and beams is given in Section 9-7 and reads as follows:

"Some vertical web reinforcing should be provided for all flexural members subjected to blast loads. A minimum of 50-psi shear stress capacity should be provided by shear steel in the form of stirrups. In those cases where analysis indicates a requirement of vertical shear reinforcing, it should be provided *in the form of stirrups.*"

TM 5-855-1 states that shear failures are unlikely in normally constructed two-way slabs, but that the possibility of shear failure increases in some protective construction

applications due to high-intensity loads. Shear is given as the governing mode of failure for deep, square, two-way slabs. For beams, one-way slabs, and two-way slabs, the manual recommends a design ductility ratio of 5.0 to 10.0 for flexural design. The recommended response limits are only given in terms of ductility ratios, not support rotations.

2.4 Army Engineer Technical Letter 1110-9-7

ETL 1110-9-7 (4) is a recent guide developed to supplement TM 5-855-1. Much of the ETL was written by this author, based on the data reviewed as part of this study; therefore, it is the result of an effort to incorporate the results of recent data into a guidance document. In brief, the criteria given in the ETL for restrained slabs allow design support rotations of 12 and 20 degrees for anticipated damage levels categorized as "moderate" and "heavy", respectively. The moderate damage level is described as that recommended for the protection of personnel and sensitive equipment. Significant concrete scabbing and reinforcement rupture have not occurred at this level. The dust and debris environment on the protected side of the slab is moderate; however, the allowable slab motions are large. Heavy damage means that the slab is at incipient failure. Under this damage level, significant reinforcement rupture has occurred, and only concrete rubble remains suspended over much of the slab. The heavy damage level is recommended for cases in which significant concrete scabbing can be tolerated, such as for the protection of water tanks and stored goods and other insensitive equipment.

The ETL sets forth some design conditions that must be

satisfied in conjunction with applying its response limits. These limitations reflect an aggressive approach, yet maintain appropriate conservatism based on available data. The scaled range must exceed 0.5 ft/lb$^{1/3}$ and the span-to-effective-depth (L/d) ratio of the slab must exceed 5. Principal reinforcement spacing is to be minimized and shall never exceed the effective depth (d) of the slab. Stirrup reinforcement is required, regardless of computed shear stress, to provide adequate concrete confinement and principal steel restraint in the large-deflection region. Stirrups are required along each principal bar at a maximum spacing of one-half the effective depth (d/2) when the scaled range is less than 2.0 ft/lb$^{1/3}$ and at a maximum spacing equal to the effective depth at larger scaled ranges. All stirrup reinforcement is to provide a minimum of 50 psi shear stress capacity.

The following types of stirrups are permitted by the ETL:

a. Single-leg stirrups having a 135-degree bend at one end and at least a 90-degree bend at the other end. When 90-degree bends are used at one end, the 90-degree bend should be placed near the compression face.

b. U-shaped and multi-leg stirrups with at least 135-degree bends at each end.

c. Closed-looped stirrups that enclose the principal reinforcement and have at least 135-degree bends at each end.

2.5 U.S. Air Force, Europe (USAFE) Semihard Design Criteria

The purpose of the document (5) is to give guidance for semi-

hardened and protected facilities with conventional, nuclear, biological, and chemical weapon protection. It states that these structures shall be designed to provide a ductile response to blast loading. Ductility of structural members is considered imperative to provide structural economy and energy absorption capability and to preclude catastrophic (brittle) failures. For design, a ductility ratio of 10 may be used, or theoretical joint rotations are to be limited to less than 4 degrees. Where explosive testing provides a sufficient data base, designers may size structural members to duplicate the performance of acceptable specimens in the data base. Structural deformations must not prohibit functional operation of the structure nor produce dangerous, high velocity, concrete spall fragments. All reinforced concrete sections are required to be doubly reinforced (reinforced in both faces) in both longitudinal and transverse directions. Where flexural response is predicted to be significant, the structural element is to be reinforced symmetrically, i.e. the compression and tension reinforcement are identical. The use of stirrups is discussed as follows:

"Ties and/or stirrups shall be provided in all members to provide concrete confinement, shear reinforcement, and to enable the element to reach its ultimate section capacity. Without stirrups, cracking and dislodgement of the concrete from between the reinforcement layers and buckling of the compression steel usually produce failure long before the ultimate strain of the reinforcement and the maximum energy absorption are attained. Stirrups contribute to the integrity of the element in the following ways:

a. The ductility of the primary flexural steel is developed.

b. Integrity of the concrete between the two layers of flexural reinforcement is maintained.

c. Compression reinforcement is restrained from buckling.

d. High shear stresses at the supports are resisted.

e. The resistance to local shear failure produced by the high intensity of the peak blast pressures is increased.

f. Quantity and velocity of post-failure fragments are reduced. Stirrups shall be bent a minimum of 135 degrees around the interior face steel and 90 degrees around the exterior face steel. Shear, splice, and anchorage details shall receive added design attention. Designers shall refer to protective design manuals and/or seismic design manuals for appropriate details."

The document does not address the use of laced reinforcement. The above list of ways that stirrups enhance the integrity of structural elements is very similar to the wording given in TM 5-1300 for the ways that lacing enhances the integrity of structural elements, except for the stirrup details given in Item f above.

2.6 Summary of Design Criteria

The criteria review indicates that guidance documents differ on the type of shear reinforcement required; however, the use of some type of shear reinforcement is uniformly required for blast

design. TM 5-1300 places restrictions on the use of slabs containing stirrups that are significantly different from those for the use of laced slabs. It allows the use of stirrups in elements designed to undergo support rotations of up to 8 degrees for scaled ranges greater than 1.0 ft/lb$^{1/3}$ when restraint against lateral movement exists at the supports. Lacing bars are required by TM 5-1300 for support rotations greater than 8 degrees and for detonations at scaled ranges less than 1.0 ft/lb$^{1/3}$. Laced slabs, whether restrained against lateral movement or not, may be designed to undergo support rotations of 12 degrees. Although TM 5-855-1 and the USAFE semihardened criteria do not require lacing, they do require some form of shear reinforcement in all elements designed to resist blast loads.

CHAPTER 3

DESCRIPTION AND DISCUSSION OF PREVIOUS EXPERIMENTAL STUDIES

3.1 Introduction

Previous experimental studies were reviewed in order to gain an understanding of their contributions to the development of current design guidance. The data review also allowed the identification of significant gaps in the data base that need to be filled to enhance further development of design criteria. This chapter presents a discussion of the available experimental data, particularly for one-way slabs, considered to be most applicable to this study. Both detailed and condensed tables containing design parameters and response values are presented. Section 3.2 describes the presentation of the data in the tables. Brief summaries of the overall purpose and results of the experimental series are presented in Section 3.3. General and detailed discussions of the data are presented in Sections 3.4 and 3.5, respectively. The chapter closes in Section 3.6 with comments regarding the application of the data to the development of design criteria.

3.2 Presentation of Data from Previous Experiments

Known construction parameters and results of the available pertinent experiments are presented in Tables 3.1 through 3.4. Figures 3.1 through 3.4 provide a means to visually evaluate the ranges of the design parameters and response values given in Tables 3.1 through 3.4. Data for a total of 258 tests are presented. Fifty-four of the tests were static loadings of one-way slabs, and ten were static loadings of box elements.

One-hundred, twenty-one of the tests were dynamic loadings of slabs, most of which were one-way slabs. Seventy-three tests were dynamic loadings of the box-type structures. The tests were conducted by the U.S. Army Engineer Waterways Experiment Station, the Air Force Armament Laboratory, the U.S. Naval Civil Engineering Laboratory, or the Picatinny Arsenal.

Data Notation

The element identification number is given in the first column of each table and usually begins with the initial of the author of the report on that particular study. The author's initial, or other descriptive letter(s), is followed by a number assigned to the specimen by that author. The identification number also includes the year that the report or paper for the experiment was published. In Table 3.3, most of the element identification numbers deviate from the form described above and contain four parts that may be described with terms used in the reports as follows:

A-B-C-D

where

A: FS (full scale); 1/3 (1/3-scale); 1/8 (1/8-scale)

B: 1 (standard slab 1)

2 (standard slab 2)

S1 (strengthened slab 1)

S2 (strengthened slab 2)

etc.

C: year of test series

D: consecutive numbering of specimens

The "restraint" column indicates the support conditions that were used. Most of the statically loaded slabs were clamped at the supports with steel plates and were considered to have rigid support conditions. The support structure of the G-84 series of statically tested slabs allowed some rotational freedom, resulting in partial restraint. The slabs of the box elements were monolithically supported either at two or at four sides by walls of the box. As defined in the legend on Table 3.3, support conditions varied the most among the dynamically loaded slabs. For many of those slabs, it is not clear as to what was the relative amount of restraint imposed by the support conditions.

Most of the dynamic slab tests were conducted by the Picatinny Arsenal. The reports on many of those tests did not present some of the parameters listed as headings in Tables 3.1 through 3.4. In particular, the effective depth (d) of the slab, the concrete compressive strength (f'_c), the steel yield strength (f_y), the spacing of the principal steel (s), and the spacing of the shear reinforcement (S_s) were often not reported. The thickness (t) of the slab was always reported. Therefore, the clear-span-to-thickness (L/t) ratio is presented in the tables rather than the more commonly used L/d ratio. Similarly, the ratios of principal steel spacing to thickness (s/t) and shear reinforcement spacing to thickness (S_s/t) are given where known. The tension steel quantity (ρ) and the compression steel quantity (ρ'), each given as a percentage of the slab width and effective depth, at the midspan and the support are reported for all slabs.

The shear reinforcement ratio (ρ_s) is also known for all slabs.

In this thesis, ρ_s is defined as the ratio of the cross-sectional area of the shear reinforcement bar (stirrup or lacing bar) to the product of the lateral spacing and the longitudinal spacing of the shear reinforcement. For all slabs discussed, the lateral spacing of the shear reinforcement is equivalent to the principal steel spacing (s), and the longitudinal spacing is equivalent to the shear reinforcement spacing (S_s). Since ρ_s is computed using the cross-sectional area of the shear reinforcement bar, the value is not affected by the inclination of the lacing bar. No inclined stirrups were used in any of the slabs discussed in this thesis.

The scaled range is presented for all dynamic tests except in the case of the HEST (High Explosive Simulation Technique) tests, for which it is not appropriate. A HEST setup consists of a cavity that is constructed above a structure, typically a buried structure. Explosives are distributed within the cavity, and a soil overburden is placed over the cavity. In general, a HEST loading results in a relatively uniform dynamic load over a large surface. The development of this procedure is discussed in detail in Reference 7. The type of reinforcing bars used for the principal steel is presented for some of the dynamic tests. Nearly all of the statically tested slabs were constructed with heat-treated deformed wire. For the static tests and a few of the dynamic tests, the support rotation (θ) at test termination or collapse is presented. The permanent deflection (Δ_{perm}) is reported for the dynamic tests when known.

The general load-deflection curve for a reinforced concrete

slab may be described as in Figure 3.5. The ultimate resistance (u) used in Tables 3.1 and 3.2 is defined by point A. The incipient failure load (I) is the load resistance occurring when the structure is about to collapse and lose its load-carrying ability. For a ductile slab experiencing tensile membrane behavior, the incipient failure load is at point C of Figure 3.5. For a brittle slab, I and u may have nearly the same value. However, I and u may have similar values in a ductile slab that experiences tensile membrane behavior. Therefore, the I/u ratio should only be examined in context with the value of support rotation, θ. The ratio I/u is presented for the static tests since the load-deflection curve is easily obtained in static tests.

The "Remarks" section of each table includes comments about special construction details and the test results. The symbols used in the remarks section as well as in some of the other columns are defined in the legends of the tables and correspond to the notation given in the reports documenting the data.

3.3 General Description of Previous Experiments

General descriptions of the previous experimental studies are given below to supplement and provide some background information for the data presented in Tables 3.1 through 3.4. The title given for each series is consistent with the element names given in the tables.

K-82 and SB-82 Series

Series:	K-82
Type:	One-way slabs
Supports:	Fixed, restrained

Loadings:	1 - static, at surface
	2 - static, buried at L/2
Flex. Steel:	ρ = 0.0050, top and bottom
Shear Steel:	ρ_s = 0.0025, closed hoops
L/t:	8.3
Agency:	WES
Reference:	8
Table:	3.1

Series:	SB-82
Type:	Box Elements, one-way action
Supports:	Fixed, restrained
Loadings:	2 - Dynamic, buried at L/2
Flex. Steel:	ρ = 0.0050, top and bottom
Shear Steel:	ρ_s = 0.0025, closed hoops
L/t:	8.3
Agency:	WES
Reference:	8
Table:	3.4

Kiger, Eagles, and Baylot (8) statically tested three one-way slabs and dynamically tested two one-way slabs as part of a study to evaluate the effects of soil cover on the capacity of earth-covered slabs. The results indicated that the capacity of the slab buried in sand was substantially greater than either the surface-flush slab or the slab buried in clay. The authors of Reference 8 attributed this increased load capacity to soil-structure interaction and used the term "soil arching." They concluded that soil arching acted to distribute much of the load from the center region of the slab to the supports.

B-83 Series

Type:	One-way slabs
Supports:	Fixed, restrained
Loadings:	3 - static, at surface
Flex. Steel:	1 - ρ = 0.0047, top and bottom
	1 - ρ = 0.0104, top and bottom
	1 - ρ = 0.0046, top and bottom
Shear Steel:	1 - ρ_s = 0.0023, single-leg stirrup
	1 - ρ_s = 0.0098, single-leg stirrup
	1 - ρ_s = 0.0041, single-leg stirrup
L/t:	2 - 10.0
	1 - 5.0
Agency:	WES

Reference:	9
Table:	3.1

Baylot and others (9) conducted three static tests on one-way slab elements as part of a program to investigate the vulnerability of buried structures to conventional weapons. Although large deflections were not achieved, the tests indicated that slabs with adequate lateral support will develop a significant enhancement in ultimate capacity due to compressive membrane action.

W-83 Series

Type:	One-way slabs
Supports:	Fixed, restrained
Loadings:	10-static, at surface
Flex. Steel:	8 - ρ = 0.0085, top; 0.0074, bottom
	2 - ρ = 0.0086, top; 0.0075, bottom
Shear Steel:	1 - None
	1 - ρ_s = 0.0009, single-leg stirrup
	5 - ρ_s = 0.0018, single-leg stirrup
	1 - ρ_s = 0.0019, single-leg stirrup
	1 - ρ_s = 0.0036, single-leg stirrup
	1 - ρ_s = 0.0038, single-leg stirrup
L/t:	10.4
Agency:	WES
Reference:	10
Table:	3.1

Woodson (10) tested ten one-way reinforced concrete slabs, primarily to investigate the effects of stirrups and stirrup details on the load response behavior of slabs. Support rotations between 13 and 21 degrees were observed. Figure 3.6 is a posttest view of the slabs. As a result of the increase in resistance with increasing deflections of a slab with a large number of single-leg stirrups, the loading of the slab was not terminated until support rotations were approximately 21 degrees (see Figure 3.7). A slab having no shear reinforcement achieved support rotations greater than 16 degrees without failure. These slabs had sufficient

lateral restraint to develop in-plane forces in the tensile membrane region of response. In this case, TM 5-1300 would allow a dynamically loaded slab with single-leg stirrups to undergo maximum support rotations up to 8 degrees. The slab with 21 degrees of support rotation contained single-leg stirrups (135-degree bend on one end and a 90-degree bend on the other end) spaced at about 0.4 d. The maximum spacing allowed in TM 5-1300 is 0.5 d, and at least 135-degree bends are required on each end of the stirrup.

W-84 Series

Type:	One-way slabs
Supports:	Fixed, restrained
Loadings:	15-Static, at surface
Flex. Steel:	1 - ρ = 0.0085, top; 0.0074 bottom
	1 - ρ = 0.0079, top and bottom
	3 - ρ = 0.0040, top; 0.0114, bottom
	1 - ρ = None, top; 0.0158, bottom
	7 - ρ = 0.0045, top; 0.0113, bottom
	1 - ρ = 0.0102, top and bottom
	1 - ρ = 0.0045, top; 0.0079, bottom
Shear Steel:	9 - None
	1 - ρ_s = 0.0006, single-leg stirrup
	4 - ρ_s = 0.0022, single-leg stirrup
	1 - ρ_s = 0.0153, single-leg stirrup
L/t:	14 - 10.4
	1 - 8.3

Woodson and Garner (11) statically tested fifteen one-way slabs to determine the effects of principal steel quantities and details on slab behavior. A posttest view of the slabs is shown in Figure 3.8. All but two of the slabs contained approximately the same total area of continuous principal steel as that of the W-83 series. However, the distribution of the total area of principal steel was varied. Reinforcement details which were investigated included the use of dowels (short lengths of reinforcement in-plane with the principal steel bars) at the

supports, the use of bent-up bars, and the use of cut-off bars (principal steel bars not extending into the supports). Duplicate slabs with bent-up bars and closely spaced stirrups were tested to indicate the repeatability of experimental results for slabs with identical construction details. All slabs were rigidly restrained at the supports and loaded with uniformly distributed pressure.

The principal reinforcement configuration that resulted in the best overall performance was a combination of bent-up and straight bars. This combination consisted of 75 percent of the total longitudinal steel being placed in the tension zones at midspan and at the supports. The single-leg stirrups were spaced at about 0.4 d. Many of the slabs in this series contained no shear reinforcement, and one slab contained only bent-up bars. Nearly all of the slabs sustained support rotations greater than 20 degrees. Except for one slab, the failure mode was primarily a 3-hinged mechanism with a compressive-membrane enhancement and an increase in load resistance in the tensile membrane region. The best tensile membrane enhancement occurred for the slab in which all principal steel consisted of bent-up bars and no stirrups were used. However, due to the lack of any confining steel, large sections of concrete fell from the slab at the locations of the steel bends as this slab responded in a 4-hinged mechanism. The series demonstrated that principal steel details significantly affect the large-deflection behavior of a one-way slab.

<u>G-84 Series</u>

Type:	One-way slabs
Supports:	Partial notational restraint, laterally restrained
Loadings:	16-Static, at surface

Flex Steel:	2 - ρ = 0.0052, top and bottom
	4 - ρ = 0.0074, top and bottom
	2 - ρ = 0.0106, top and bottom
	2 - ρ = 0.0058, top and bottom
	4 - ρ = 0.0114, top and bottom
	2 - ρ = 0.0147, top and bottom
Shear Steel:	10 - ρ_s = 0.0018, single-leg stirrup
	2 - ρ_s = 0.0022, single-leg stirrup
	2 - ρ_s = 0.0024, single-leg stirrup
	2 - ρ_s = 0.0027, single-leg stirrup
L/t:	8 - 10.4
	8 - 14.8
Agency:	WES
Reference:	12
Table:	3.1

Guice (12) statically tested sixteen one-way reinforced concrete slabs with uniformly distributed load, primarily to investigate the effects of edge restraint on slab behavior. Each slab contained single-leg stirrups spaced at approximately 1.5 d. The stirrups had a 135 degree bend on one end and a 90 degree bends on the other end. Support rotations of about 20 degrees were sustained. Regardless of support rotational freedom, the tests indicated that the percentage of load carried by tensile membrane action is related to the slab's span-to-thickness ratio. Guice concluded that elements which have a span-to-thickness ratio of about 15, have 1.0 to 1.5 percent of steel in each face, and are supported with a relatively large lateral stiffness and a moderate rotational stiffness will probably result in a structure which best combines the characteristics of strength, ductility, and economy.

<u>K4S-69 and K4D-69 Series</u>

Series:	K4S-69
Type:	One-way slab
Supports:	Fixed, restrained
Loadings:	1 - Static, at surface
Flex. Steel:	ρ = 0.0211, top and bottom
Shear Steel:	ρ_s = 0.0137, lacing

L/t:	12
Agency:	NCEL
Reference:	13
Table:	3.1

Series:	K4D-69
Type:	One-way slabs
Supports:	Fixed, restrained
Loadings:	3 - Dynamic, at surface
Flex. Steel:	ρ = 0.0211, top and bottom
Shear Steel:	ρ_s = 0.0137, lacing
L/t:	12
Agency:	NCEL
Reference:	13
Table:	3.3

Keenan (13) tested four laced reinforced concrete one-way slabs. One slab was tested with an increasing static load applied by water pressure, and the other three slabs were subjected to two or more short-duration dynamic loads. Keenan reported that the rotational capacity at the critical sections of the statically tested slab was greater than 9.2 degrees, but could not be exactly determined due to safety limitations on the loading device that prohibited further response. Slab behavior was similar under static and dynamic load. Keenan stated that the type of loading did not change the extent of cracked or crushed concrete, the collapse mechanism, the mode of failure, or the rotational capacity at supports. He reported that the stress in the lacing bars at the hinges was induced by rotation of the cross-section in addition to shear. The tests showed that the effects of rotation, in addition to shear, should be considered in designing lacing reinforcement for sections near a support.

<u>K9S-69 and K9D-69 Series</u>

Series:	K9S-69
Type:	Two-way slabs
Supports:	Fixed, restrained
Loadings:	6 - Static, at surface

Flex. Steel:	1 - None
	3 - ρ = 0.0082, top and bottom
	1 - ρ = 0.0089, top and bottom
	1 - ρ = 0.0133, top and bottom
Shear Steel:	1 - None
	3 - ρ_s = 0.0019, lacing
	1 - ρ_s = 0.0042, lacing
	1 - ρ_s = 0.0167, lacing
L/t:	4 - 24
	1 - 15.2
	1 - 12
Agency:	NCEL
Reference:	14
Table:	3.1

Series:	K9D-69
Type:	Two-way slabs
Supports:	Fixed, restrained
Loadings:	3 - Dynamic, at surface
Flex. Steel:	1 - ρ = 0.0082, top and bottom
	2 - ρ = 0.0089, top and bottom
Shear Steel:	1 - ρ_s = 0.0019, lacing
	2 - ρ_s = 0.0042, lacing
L/t:	1 - 24
	2 - 15.2
Agency:	NCEL
Reference:	14
Table:	3.3

Keenan (14) tested nine reinforced concrete two-way slabs. The slabs were square and restrained against rotation and lateral movement at the edges. Keenan discussed the observation of "tensile-membrane fragments" that were the size of the reinforcing mesh spacing in a slab that contained no lacing at midspan. This slab only had lacing near the supports and contained no stirrups. It was observed that lacing at midspan prevented this type of fragmentation in another slab. However, lacing did not prevent severe spalling. It was concluded that slabs should contain lacing or closely spaced principal reinforcement to prevent fragmentation caused by dynamic deflections in the tensile membrane region of behavior. None of the slabs contained stirrups.

Although TM 5-1300 does not address the use of closely spaced principal reinforcement, test data indicate that using smaller principal reinforcing bars with a reduced spacing will enhance the ductile response of slabs. This was reported by Keenan (13 and 14) and Woodson (10).

K-78,79 and FH-78,79 Series

Series:	K - 78,79
Type:	Box structures, one-way action
Supports:	Fixed, restrained
Loadings:	2 - Static, buried at L/2
	2 - Static, buried at L/5
Flex. Steel:	3 - ρ = 0.0100, top and bottom
	1 - ρ = 0.0185, top and bottom
Shear Steel:	3 - ρ_s = 0.0153, single-leg stirrup
	1 - ρ_s = 0.0110, single-leg stirrup
L/t:	3 - 8.3
	1 - 3.3
Agency:	WES
References:	15, 17, 18
Table:	3.2

Series:	FH - 78,79
Type:	Box structures, one-way action
Supports:	Fixed, restrained
Loadings:	4 - Dynamic, buried at L/2
	3 - Dynamic, buried at L/5
Flex. Steel:	6 - ρ = 0.0100, top and bottom
	1 - ρ = 0.0150, top and bottom
Shear Steel:	1 - ρ_s = 0.0150, double-leg stirrup
	6 - ρ_s = 0.0150, double-leg stirrup
L/t:	8.6
Agency:	WES
References:	16-20
Table:	3.4

Kiger and Getchell (15 through 20) conducted seven dynamic tests and four static tests investigating the effects of load intensity, backfill type, and depth-of-burial on the response of one-way roof slabs of box elements. The dynamic tests were conducted with 1/4-scale box structures loaded by simulated nuclear overpressures utilizing a HEST. The static tests were

conducted on 1/8-scale structures in the Large Blast Load Generator at WES. Huff (21) describes the capabilities of the test device.

Figure 3.9 shows the damage incurred by a box structure (FH3-78) buried 2 feet deep in clay and subjected to a simulated nuclear overpressure of approximately 2000 psi peak pressure. Permanent deflection was at approximately 6 inches (corresponding to approximately 14 degrees support rotation) with some concrete cover broken free. In another experiment (FH4-79), a box was buried 10 inches in sand and also loaded with approximately 2000 psi peak pressure. Figures 3.10 and 3.11 show a partial failure of the roof and some loss of concrete cover from the reinforcement. Permanent roof deflections were approximately 12.5 inches (corresponding to approximately 28 degrees support rotation). Although the roof was clearly on the verge of collapse, it did sustain this level of damage at a very high pressure without catastrophic failure.

<u>S-83, F-83, and F-84 Series</u>

Series:	S-83
Type:	Box-elements, one-way action
Supports:	Fixed, restrained
Loadings:	1 - Static, at surface
	5 - Static, buried at 4L/11
Flex. Steel:	ρ = 0.0069, top and bottom
Shear Steel:	ρ_s = 0.0018, double-leg stirrup
L/t:	13.2
Agency:	WES
Reference:	22
Table:	3.2

Series:	F-83
Type:	Box-elements, one-way action
Supports:	Fixed, restrained
Loadings:	1 - Dynamic, at surface
	7 - Dynamic, buried at 4L/11
Agency:	WES

Reference:	22
Table:	3.4

Series:	F-84
Type:	Box elements, one-way action
Supports:	Fixed, restrained
Loadings:	4 - Dynamic, buried at 4L/11
Flex. Steel:	ρ = 0.0040, top; 0.0120, bottom
Shear Steel:	None
L/t:	14.7
Agency:	WES
Reference:	22
Table:	3.4

Slawson and others (22) conducted six static and twelve (four were repeated dynamic loads) dynamic tests investigating structural design, structural response in various backfills, the effects of concrete strength on response, and the effects of repeated loadings on structural response. The slabs contained single-leg stirrups at a moderate spacing and most of the roof slabs in the static tests sustained support rotations greater than 15 degrees.

FS-1-63 and 1/3-1-63 Series

Series:	FS-1-63
Type:	Two-way slabs, full-scale
Supports:	Simple, unrestrained
Loadings:	5 - Dynamic, at surface
Flex. Steel:	ρ = 0.0015, top and bottom
Shear Steel:	None
L/t:	8
Agency:	Picatinny Arsenal
Reference:	23
Table:	3.3

Series:	1/3-1-63
Type:	Two-way slabs, 1/3-scale
Supports:	Simple, unrestrained
Loadings:	6 - Dynamic, at surface
Flex. Steel:	ρ = 0.0015, top and bottom
Shear Steel:	None
L/t:	8
Agency:	Picatinny Arsenal
Reference:	23
Table:	3.3

Rindner and Schwartz (23) summarized tests conducted up through December, 1964, in support of the establishment of design criteria for facilities used for operations dealing with explosives. Eleven dynamic tests were conducted primarily to investigate the validity of scale-model testing. The slabs were tested in a horizontal position, resting on timber supports on the ground. The range of damage extended from surface pitting to complete destruction, producing rubble. In most of the tests, the supporting timbers were displaced and severely damaged. Donor charges were placed at various standoff distances and consisted of bare cylinders of Composition B for the smaller charges, but the explosive was encased in 1/8-thick pipe for the larger charges. The study showed a good qualitative correlation of damage between the experiments using the full-scale and 1/3-scale models under similar loading and support conditions. None of the slabs contained any shear reinforcement, and all contained only about 0.15 percent principal reinforcement in each face. The scaled ranges varied from approximately 1.0 to 2.6 $ft/lb^{1/3}$.

<u>FS-1-64 and 1/3-1-64 Series</u>

Series:	FS-1-64
Type:	Two-way slabs, full-scale
Supports:	1 - Simple, unrestrained
	2 - Fixed, restrained
Loadings:	3 - Dynamic, at surface
Flex. Steel:	ρ = 0.0015, top and bottom
Shear Steel:	None
L/t:	8
Agency:	Picatinny Arsenal
Reference:	23
Table:	3.3

Series:	1/3-1-64
Type:	Two-way slabs, 1/3-scale
Supports:	1 - Simple, unrestrained
	2 - Fixed, restrained

Loadings:	3 - Dynamic, at surface
Flex. Steel:	ρ = 0.0015, top and bottom
Shear Steel:	None
L/t:	8
Agency:	Picatinny Arsenal
Reference:	23
Table:	3.3

Rindner and Schwartz (23) also summarized a second series of scaling investigations. Six slabs were tested to further investigate the feasibility of one-third scale testing and to investigate different methods of slab support that would allow photographic coverage of slab fragment movement. Four of the slabs were supported by structural steel frames. The supports were destroyed by blasts in the vertical tests of the series. None of the slabs contained shear reinforcement and scaled distances varied from approximately 1.0 to 2.6 ft/lb$^{1/3}$. Slab damage ranged from surface cracking to break-up of the slab into a few sections. The one-third scale slabs displayed brittle failure characteristics while the full-scale slabs tended to crack and deflect.

CAM-64 Series

Type:	Two-way slabs
Supports:	2 - Simple, unrestrained 1 - Fixed, restrained
Loadings:	3 - Dynamic, at surface
Flex. Steel:	ρ = 0.0015, top and bottom
Shear Steel:	None
L/t:	8
Agency:	Picatinny Arsenal
Reference:	23
Table:	3.3

Rindner and Schwartz (23) also included discussion of three tests that were conducted to further investigate methods of slab support that would allow photographic coverage of slab fragment movement. Two of the slabs were supported in a horizontal

position on heavy steel plates on edge. The third slab was supported in a vertical position by walls of a steel tunnel. None of the slabs contained any shear reinforcement, and scaled ranges were approximately 0.5 ft/lb$^{1/3}$ in each test. Each slab was completely destroyed.

BAL-64 Series

Type:	Two-way slabs
Supports:	Fixed, restrained
Loadings:	2 - Dynamic, at surface
Flex. Steel:	ρ = 0.0130, top and bottom
Shear Steel:	None
L/t:	8
Agency:	Picatinny Arsenal
Reference:	23
Table:	3.3

In these two experiments, presented by Rindner and Schwartz (23), slabs were constructed with balanced steel percentages of approximately 1.3 percent in each face. No shear reinforcement was used. One slab was tested at a scaled range of 0.5 ft/lb$^{1/3}$, and one was at 2.5 ft/lb$^{1/3}$. For the scaled range of 0.5 ft/lb$^{1/3}$, the slab was reduced to small rubble. For the scaled range of 2.5 ft/lb$^{1/3}$, the slab experienced heavy damage with large cracks and some rubble.

1/3-2-64 and 1/3-S1-64 Series

Series:	1/3-2-64
Type:	Two-way slabs, 1/3 scale
Supports:	Fixed, restrained
Loadings:	5 - Dynamic, at surface
Flex. Steel:	ρ = 0.0015, top and bottom
Shear Steel:	None
L/t:	8
Agency:	Picatinny Arsenal
Reference:	23
Table:	3.3

Series:	1/3-S1-64
Type:	Two-way slabs, 1/3-scale

Supports:	Fixed, restrained
Loadings:	4 - Dynamic, at surface
Flex. Steel:	ρ = 0.0040, top and bottom
Shear Steel:	None
L/t:	14
Agency:	Picatinny Arsenal
Reference:	23
Table:	3.3

These tests were also summarized by Rindner and Schwartz (23) and were conducted to investigate the responses of various basic types of slabs when subjected to different loading conditions. Scaled ranges varied from approximately 0.5 to 3.5 ft/lb$^{1/3}$. The extent of the damage ranged from hairline cracks to complete destruction.

<u>1/3-65 Series</u>

Type:	Two-way slabs, 1/3-scale
Supports:	22 - Fixed, restrained
	9 - Fixed, unrestrained
Loadings:	31 - Dynamic, at surface
Flex. Steel:	1 - ρ = 0.0015, top and bottom
	2 - ρ = 0.0044, top and bottom
	5 - ρ = 0.0065, top and bottom
	1 - ρ = 0.0075, top and bottom
	17 - ρ = 0.0140, top; 0.0065, bottom
	1 - ρ = 0.0027, top; 0.0065, bottom
	1 - ρ = 0.0133, top; 0.0069, bottom
	3 - ρ = 0.0270, top and bottom
Shear Steel:	20 - None
	1 - ρ_s = 0.0003, loop
	2 - ρ_s = 0.0015, lacing
	4 - ρ_s = 0.0040, lacing
	2 - ρ_s = 0.0053, lacing
	2 - ρ_s = 0.0120, lacing
L/t:	1 - 1.85
	2 - 2
	1 - 4
	27 - 6
Agency:	Picatinny Arsenal
Reference:	24
Table:	3.3

Rindner, Wachtell, and Saffian (24) summarized tests conducted during 1965 for the establishment of design criteria. Thirty-one tests conducted in that year are applicable to this

study. The tests were conducted to:

a. establish the explosive quantity range for specially reinforced concrete

b. establish a general configuration of reinforced concrete (plain, composite, etc.) which will be used in the construction of explosive facilities

c. evaluate the blast loading (impulse) applied to the wall

d. investigate the optimum amount of reinforcement and the maximum amount of reinforcement that is feasible in the construction of explosive-storage cubicles

e. evaluate specific detailing of reinforcement (various types of shear reinforcement and placement of reinforcement).

Most of the slabs contained no shear reinforcement, but ten slabs contained lacing. One slab contained "looped" shear reinforcement. Scaled ranges varied from approximately 0.4 to 1.6 $ft/lb^{1/3}$. The slabs were either supported in the steel tunnel or in the "new support structure" designed for charges over 30 lbs. Bending-restraint plates were also used in some of the tests, but those particular slabs were not laterally restrained. It was concluded that a substantial increase in slab capacity is accomplished by strengthening the slab (using a higher percentage of reinforcement) and by the proper use of ties (shear reinforcing in the form of lacing) which significantly increased the resistance to blast.

1/3-66 and 1/8-66 Series

Series:	1/3-66
Type:	Two-way slabs, 1/3-scale
Supports:	Fixed, restrained
Loadings:	13 - Dynamic, at surface
Flex. Steel:	7 - ρ = 0.0065, top and bottom
	6 - ρ = 0.0200, top and bottom
Shear Steel:	6 - ρ_s = 0.0015, lacing
	1 - ρ_s = 0.0030, loop
	6 - ρ_s = 0.0120, lacing
L/t:	1 - 2
	6 - 4
	6 - 6
Agency:	Picatinny Arsenal
Reference:	25
Table:	3.3

Series:	1/8-66
Type:	Two-way slabs, 1/8-scale
Supports:	Fixed, restrained
Loadings:	15 - Dynamic, at surface
Flex. Steel:	1 - ρ = 0.0015, top and bottom
	1 - ρ = 0.0065, top and bottom
	10 - ρ = 0.0140, top; 0.0065 bottom
	3 - ρ = 0.0270, top and bottom
Shear Steel:	6 - None
	1 - ρ_s = 0.0015, lacing
	5 - ρ_s = 0.0040, lacing
	3 - ρ_s = 0.0120, lacing
L/t:	3 - 4
	12 - 6
Agency:	Picatinny Arsenal
Reference:	25
Table:	3.3

Rindner, Wachtell, and Saffian (25) discussed this series conducted in 1966 to:

a. determine both qualitative and quantitative data on slab response

b. investigate the effects of high and low compression strength concrete and the addition of fibrous materials (cut wire and nylon).

c. determine the validity of 1/8-scale testing.

Most of the slabs contained lacing. One slab contained looped reinforcement, and six slabs had no shear reinforcement.

Scaled ranges varied from 0.3 to 1.25 lb/ft$^{1/3}$. Damage levels ranged from slight damage to total destruction.

<u>1/3-67 Series</u>

Type:	Two-way slabs, 1/3-scale
Supports:	Fixed, restrained
Loadings:	19 - Dynamic, at surface
Flex. Steel:	5 - ρ = 0.0065, top and bottom
	14 - ρ = 0.0270, top and bottom
Shear Steel:	5 - ρ_s = 0.0015, lacing
	14 - ρ_s = 0.0120, lacing
L/t:	3 - 2
	5 - 4
	11 - 6
Agency:	Picatinny Arsenal
Reference:	26
Table:	3.3

Rindner, Wachtell, and Saffian (26) summarized tests conducted during 1967 for the establishment of design criteria. All of the slabs were bolted into the "modified new support structure" which included the use of lateral restraining plates. All of the slabs contained laced reinforcement, and scaled ranges varied from 0.50 to 1.65 ft/lb$^{1/3}$. The slabs were tested to obtain data for the design of reinforced concrete laced elements subjected to close-in blasts. The tests also evaluated the use of fibrous reinforced concrete for reducing spall and the use of low compressive strength concrete (2,500-3,000 psi).

It was concluded that the impulse capacity of reinforced slabs containing fibers is larger than that of slabs without fibrous material. There was no significant loss in capacity due to the reduced concrete strength. It was concluded that incipient failure of a laced reinforced concrete element may be described by a maximum deflection corresponding to a support rotation of 12 degrees.

T-88 Series

Type:	Two-way slabs
Supports:	Fixed, restrained
Loadings:	6 - Dynamic, at surface
Flex. Steel:	1 - ρ = 0.0031, top and bottom
	3 - ρ = 0.0100, top and bottom
	1 - ρ = 0.0150, top and bottom
	1 - ρ = 0.0250, top and bottom
Shear Steel:	1 - None
	1 - ρ_s = 0.0022, lacing
	1 - ρ_s = 0.0045, single-leg stirrup
	1 - ρ_s = 0.0047, single-leg stirrup
	1 - ρ_s = 0.0048, single-leg stirrup
	1 - ρ_s = 0.0049, single-leg stirrup
L/t:	1 - 15
	5 - 20
Agency:	NCEL
Reference:	27
Table:	3.3

Tancreto (27) tested six two-way slabs to verify design criteria for slabs with tensile membrane resistance and to investigate the effects of stirrup details on the response of reinforced concrete slabs at large support rotations (described as being greater than 4 degrees) and for close-in explosions. The slabs were not loaded to failure. The tests indicated that the TM 5-1300 breaching criterion is conservative since stirrups were adequate at a scaled range of 0.7 ft/lb$^{1/3}$, which is less than the value of 1.0 lb/ft$^{1/3}$ specified in TM 5-1300 as the lower limit for the use of stirrups. Stirrup spacings equal to the slab effective depths were described as being adequate, as opposed to the upper limit of d/2 given in TM 5-1300. Tancreto concluded that more tests are needed to establish:

a. improved breaching criteria

b. allowable stirrup spacing (for flexural ductility and for shear)

c. allowable maximum rotation for slabs containing

stirrups

d. ultimate rotation with tensile membrane resistance.

DS-81 and DS-82 Series

Series:	DS-81
Type:	Box elements, one-way action
Supports:	Fixed, restrained
Loadings:	5 - Dynamic, buried at L/5
Flex. Steel:	ρ = 0.0100, top and bottom
Shear Steel:	1 - ρ_s = 0.0150, double-leg stirrup
	4 - ρ_s = 0.0150, single-leg stirrup
L/t:	8.6
Agency:	WES
Reference:	28
Table:	3.4

Series:	DS-82
Type:	Box elements, one-way action
Supports:	Fixed, restrained
Loadings:	6 - Dynamic, buried at L/5
Flex. Steel:	3 - ρ = 0.0075, top and bottom
	3 - ρ = 0.0120, top and bottom
Shear Steel:	6 - ρ_s = 0.0050, single-leg stirrups
L/t:	6.2
Agency:	WES
Reference:	28
Table:	3.4

Slawson (28) dynamically tested eleven shallow-buried reinforced concrete box elements, primarily to evaluate dynamic shear failure criteria. The structures were subjected to high-pressure (greater than 2000 psi peak pressure) short-duration loads. Shear reinforcement consisted of single-leg stirrups with a 90-degree bend and a 135-degree bend. When what appeared to be dynamic shear failure occurred, severing the roof slab from the walls, the concrete throughout the slab was severely crushed and fell from the roof slab reinforcement mats when lifted from the floor for post-test examination.

The one-way roof slabs of four of Slawson's structures did not experience total collapse. One of these roof slabs, having a

span-to-thickness ratio of 8.6, experienced a deflection at midspan of approximately 10 inches for the 48-inch clear span. This deflection corresponds to a support rotation of approximately 23 degrees. Some spalling occurred at the walls, but the rest of the slab was cracked without spalling action (see Figure 3.12). This slab contained single-leg stirrups spaced at approximately 0.8 d with two stirrups at each location. The remaining three slabs contained one single-leg stirrup at each location, and the spacing varied from approximately 0.25 d near the supports to 0.5 d at midspan. These slabs had span-to-thickness ratios of 6.2. One slab responded predominantly in shear with a permanent midspan deflection of approximately 4.5 inches. The unloaded face of the slab experienced cracking with severe crushing of the concrete occurring only at the supports. Another roof slab experienced a midspan deflection of approximately 12 inches (corresponds to a support rotation of approximately 26 degrees). The concrete cover spalled, and the concrete between the principal reinforcement mats was broken up over the entire span but did not fall from the reinforcement cage (see Figure 3.13). These data indicated that slabs with single-leg stirrups can resist high-pressure short-duration loads without total collapse.

1/8-MC-71 Test

Type:	Box structure, two-way action
Supports:	Fixed, restrained
Loadings:	1 - Dynamic, at surface
Flex. Steel:	ρ = 0.0042, top and bottom
Shear Steel:	Unreported quantity, lacing
L/t:	10
Agency:	Picatinny Arsenal
Reference:	29
Table:	3.4

Levy and others (29) discussed a test on an 1/8-scale model cubicle wall, loaded at a scaled range of 0.5 ft/lb$^{1/3}$. The structure successfully withstood the loading with heavy damage but without failure of any reinforcement.

B-84 Series

Type:	Box structure, one-way action
Supports:	Fixed, restrained
Loadings:	1 - Dynamic, at surface
Flex. Steel:	ρ = 0.0051, top and bottom
Shear Steel:	ρ_s = 0.0003, single-leg stirrup
L/t:	14.8
Agency:	WES
Reference:	30
Table:	3.4

Baylot (30) dynamically loaded a 1/4-scale reinforced concrete model of a weapon storage cubicle using a HEST. Three layers of reinforcement were provided in the principal direction in the long walls, roof, and floor, while two layers were provided in the transverse direction. One of the three layers was placed near the center of the element's cross section. The shear steel only existed in the roof slab near the supports. The stirrups had a 135-degree bend at one end and a 90-degree bend at the other.

A 2.5 kiloton weapon with a peak pressure of approximately 1500 psi was simulated with the HEST. The midspan deflection of the roof slab was approximately 11.4 inches, corresponding to a support rotation of approximately 16 degrees. Some stirrups along the exterior wall were broken. A very small shallow zone of concrete crushing occurred down the center of the top surface of the roof slab. The largest crack on the bottom surface was approximately 1/8-inch wide.

KW-87 Test

Type:	Box structure, one-way action
Supports:	Fixed, restrained
Loadings:	1 - Dynamic, buried at L/2.75
Flex. Steel:	ρ = 0.0036, top; 0.0110, bottom
Shear Steel:	None
L/t:	12.9
Agency:	WES
Reference:	31
Table:	3.4

A full-scale 100-man capacity blast shelter was tested in a simulated nuclear overpressure environment as reported by Slawson (31). The 3-bay structure had a roof span of approximately 11 feet for each bay, a roof thickness of approximately 10.25 inches. Some principal steel (25 percent of the total) was "draped" (actually, bent-up bars were used) so that it served as tensile reinforcement at both the supports (top face) and midspan (bottom face) of the roof. No shear reinforcement was used in the roof, and the bottom face of the roof slab was covered by corrugated sheet metal that served as form work and effectively prevented any separation of the concrete from the roof that might would have occurred due to spalling action or scabbing. A posttest view of the interior of Bay 1 is shown in Figure 3.14. The maximum roof deflection was approximately 17 inches (corresponding to a support rotation of approximately 14 degrees).

F-77 Series

Type:	Box elements (walls), 3 - two-way action 20 - one-way action
Supports:	Fixed, restrained
Loadings:	23 - Dynamic, buried wall
Flex. Steel:	ρ = 0.0200, top and bottom
Shear Steel:	None
L/t:	4 - 6 6 - 9 7 - 12 6 - 18

Agency:	Air Force
Reference:	32
Table:	3.4

Fuehrer and Keeser (32) conducted a test program to provide data defining the vulnerability of underground reinforced concrete targets. The objective was to generate experimental data relating the maximum distances at which explosive charges of specified weights are capable of breaching reinforced concrete slabs. Charge weights ranged from 4.6 to 27 pounds. The maximum standoff distance at which the slabs were breached increased with decreasing values of span-to-thickness ratios.

B-85 and H-89 Series

Series:	B-85
Type:	Box elements (walls), one-way action
Supports:	Fixed, restrained
Loadings:	11 - Dynamic, buried wall
Flex. Steel:	9 - ρ = 0.0050, top and bottom
	2 - ρ = 0.0100, top and bottom
Shear Steel:	2 - ρ_s = 0.0027, single-leg stirrup
	7 - ρ_s = 0.0032, single-leg stirrup
	2 - ρ_s = 0.0050, single-leg stirrup
L/t:	2 - 5
	9 - 10
Agency:	WES
Reference:	33
Table:	3.4

Series:	H-89
Type:	Box elements (walls), one-way action
Supports:	Fixed, restrained
Loadings:	4 - Dynamic, buried wall
Flex. Steel:	1 - ρ = 0.0050, top and bottom
	3 - ρ = 0.0100, top and bottom
Shear Steel:	1 - ρ_s = 0.0028, single-leg stirrup
	3 - ρ_s = 0.0050, single-leg stirrup
L/t:	1 - 5
	3 - 10
Agency:	WES
Reference:	33
Table:	3.4

Eleven tests were conducted in the B-85 series (33) to study

the response of structures buried in sand to the loading from a point-source detonation. Each test involved a reinforced concrete slab and a cylindrical cased charge. The parameters that were varied included the charge orientation, standoff distance, span-to-thickness ratio, and the amount of reinforcing steel in the test slab.

The H-89 series (33) was conducted to investigate the effects of backfill type as a follow-up to the B-85 series. A breach occurred in a slab tested in the low-shear-strength, low-seismic-velocity, reconstituted clay backfill. Light damage occurred in a slab tested in the high-shear-strength, low-seismic-velocity sand backfill.

3.4 General Discussion of Results of Previous Experiments

The discussion that follows highlights significant features of the presented data and prepares the reader for the more detailed discussion of Section 3.5. All of the statically tested slabs were laterally restrained such that compressive and tensile membrane forces could be developed. However, as noted by Guice (12), slabs of the G-84 series that had relatively large values of rotational freedom were not able to achieve their potential compressive membrane capacity because of large, early support rotations. Therefore, the slab snapped through to the tensile membrane stage before significant thrusts were developed. For the thinner slabs of the G-84 series, this snap-through occurred for smaller rotational freedoms than for that of the thicker slabs. Small rotational freedoms at the supports, as opposed to rigid supports, enhanced the tensile membrane capacity and the incipient

collapse deflection of the slabs.

The L/t values for all of the statically tested slabs were large enough to insure that the slabs were not "deep" slabs, and that a flexural response mode was probable. All of the statically tested slabs had nearly equal percentages of steel in the top and bottom faces except for the W-84 series. The objective of that series was to investigate the effects of varying the placement of the principal steel between the compression and tension faces of the slab, while maintaining the total amount of principal steel at an equal value in all slabs. It was found that ductility increased when more of the total area of principal reinforcement was placed in the tension zones. The compressive strength of the concrete for the statically tested slabs ranged from about 3.6 to 5 ksi except for the K-82 and B-83 series, where values from 6.1 to 6.9 ksi were reported. The yield strength of the principal steel was also greater for these two series as it ranged from approximately 70 to 90 ksi. Additionally, all but one of the slabs of the K-82 and B-83 series had principal steel quantities of around 0.5 percent, compared to about 0.75 to 1.6 percent for slabs in the other static test series. Ignoring the two slabs of the K-82 series with soil cover, the slabs of these two series were similar to the other statically tested slabs for all other parameters; yet, these slabs failed at relatively small support rotations. The static slab tests of Table 3.1 demonstrated that slabs with single-leg stirrups (or even no shear reinforcement) can achieve large support rotations without collapse.

The static box tests of Table 3.2 were each tested in a buried configuration. Values of construction parameters were in

the same general range of those for the statically tested slabs of Table 3.1. One box (K4-79) had a L/t ratio of only 3.3 and failed in shear without rupture of any reinforcement. Large support rotations were achieved in many of the static box tests, all of which contained single- or double-leg stirrups.

The largest group of tests is that of the dynamically tested slabs presented in Table 3.3. Most of these tests were conducted in the 1960's with the objective of developing design criteria for the 1969 version of TM 5-1300. Most of the slabs identified in Table 3.3 contained either laced reinforcement or no shear reinforcement. Only two slabs (1/3-S12-65-1 and 1/3-S12-66-1) contained a form of stirrups (actually referred to as "looped" reinforcement). Therefore, it is not surprising that the 1969 version of TM 5-1300 imposed significant limitations on slabs with stirrups - little data was available for slabs with stirrups. Of those two slabs with looped reinforcement, one was tested at a scaled range of 1.25 ft/lb$^{1/3}$ and experienced only medium damage with no rupture of reinforcement (the revised TM 5-1300 requires lacing when the scaled range is less than 1 [illegible] ft/lb$^{1/3}$). The other slab with looped reinforcement was tested at a scaled range of 1.0 ft/lb$^{1/3}$ and was described as incurring partial destruction with all tension steel failing and with shear failure in the concrete. This slab was not laterally restrained; therefore, tensile membrane forces could not be developed. Both of these slabs had a L/t ratio of 6.0, which is near that of a deep slab where large-deflection ductile behavior is less likely to occur for moderately reinforced slabs. Due to the combinations of the construction/test parameters involved, these two slabs contributed

little to the large-felfection design criteria of TM 5-1300.

Principal steel quantities varied considerably among the dynamically tested slabs. Slab 1/3-S14-65-1 contained a large percentage of steel in each face (2.7 percent), but it did not contain shear reinforcement. The L/t ratio was equal to 4, and it was tested at a scaled range of 0.5. The slab experienced only medium damage with all steel intact. A laced slab (1/3-S13-65-1) with the same parameter values, except for L/t equal to 6, incurred heavy damage with tension steel failing at the supports and at midspan. Apparently, characteristics of shear reinforcement was not the controlling parameters affecting the response of the two slabs.

Some of the dynamically tested slabs with no shear reinforcement failed in large sections, as opposed to being reduced to "small rubble", the failure mode specified in TM 5-1300 for slabs subjected to close-in blasts. For example, slab 1/3-1-63-5 was tested at a scaled range of 0.99 $ft/lb^{1/3}$ with L/t equal to 8 and was broken into 2 large sections. Three of the slabs with no shear reinforcement were tested at a scaled range of 0.80 $ft/lb^{1/3}$. The rest of these slabs were tested at a scaled range of 1 $ft/lb^{1/3}$ or greater, or at the smaller value of approximately 0.5 $ft/lb^{1/3}$. The three slabs tested at a scaled range of 0.80 $ft/lb^{1/3}$ had a L/t value of 6 experienced total destruction. However, the slabs were unusual as they contained over twice as much compression steel as tension steel at midspan and vice versa at the supports.

Some laced slabs also experienced heavy damage. It is obvious that the laced slabs generally responded better than the

slabs with no shear reinforcement, but the limits of response for slabs without shear reinforcement cannot be determined from these tests. Additionally, the group of dynamically tested slabs makes almost no contribution to the understanding of the behavior of slabs containing stirrups.

The T-88 series is the only set of dynamic slab tests which was directed toward some comparison of laced and stirrup slabs. Only one of these six slabs contained lacing, and one contained no shear reinforcement. The slabs were not tested to failure and many parameters were varied, making it difficult to quantify the relative effectiveness of lacing and stirrups. However, the tests indicated that slabs with stirrups can achieve support rotations greater than those allowed by TM 5-1300. These slabs were two-way slabs with large L/t ratios of 15 or 20. Tancreto (27) concluded that more research is needed to determine the rotational capacity and tensile membrane behavior of slabs with stirrups, the allowable stirrup spacing, and to improve breaching criteria.

All but the roof slab of one (1/8-MC-71) of the dynamically tested boxes listed in Table 3.4 were one-way slabs and were part of the same research programs as the static tests. The boxes contained either stirrups or no shear reinforcement, and construction parameters were similar to those of the static tests. Of these dynamically tested boxes, only element F2-83 was tested at surface flush. The other boxes were buried. The 1/8-MC-71 roof slab was a two-way slab with lacing and no soil cover. It was also part of the only box that was not tested in a HEST configuration. The scaled range was 0.5 for this box, and it experienced heavy damage but no reinforcement was ruptured.

3.5 Detailed Discussion of Previous Experiments

General

The discussion in this section makes specific comparisons of the responses of slabs with various construction parameters to provide insight into the role of the parameters and to emphasize the existence of gaps in the data base. Three catergories are provided for the discussion: laterally-restrained boxes, laterally-restrained slabs, and laterally-unrestrained slabs. Selected parameters from the data base are given in Tables 3.5 through 3.10 for convenience. The following discussions refer to the parameters that are included in Tables 3.5 through 3.10.

Laterally-Restrained Boxes

The roof, floor, and wall slabs of protective structures, particularly those in the data base, are generally laterally restrained. This is partly due to the extension of the principal reinforcement of a slab into the adjoining slab. Lateral restraint is necessary for the formation of tension membrane forces that enhance the large-deflection behavior of slabs.

Parameters for boxes loaded with point-source charges are presented in Table 3.5. Most of the boxes were tested at scaled ranges of 2.0 $ft/lb^{1/3}$ or less, were buried, and had a tension reinforcement quantity equivalent to 2.0 percent. For slabs of boxes tested at a scaled range of 1.0 $ft/lb^{1/3}$ and having low values of L/t in the range of approximately 5 or 6, damage was slight, and support rotations were small (5 to 7 degrees). Some wall slabs of boxes having L/t values of approximately 8 to 12 experienced large support rotations (15 to 29 degrees) and were

damaged to near incipient collapse. However, a wall slab with a small L/t value equal to 6 was tested at a scaled range of 0.75 $ft/lb^{1/3}$ and sustained a support rotation of 26 degrees without breaching, although it contained no shear reinforcement. Breaching did not occur in the group of slabs tested at scaled ranges less than 2.0 $ft/lb^{1/3}$ until support rotations reached 15 degrees, and some slabs achieved support rotations significantly greater that 15 degrees without breaching occurring. In general, no shear reinforcement was used in this group of slabs.

The data base also includes a group of laterally-restrained slabs (components of box structures) tested at a scaled range of 2.0 $ft/lb^{1/3}$ or greater. The L/t values for these slabs ranged from approximately 5 to 18 and ρ was relatively large, 2.0 percent (the upper limit allowed by TM 5-855-1 for ductility considerations). Although support rotations were generally small and the damage was slight (mainly hairline cracks), support rotations were as large as 26 degrees for a wall slab (L/t of 10) of a box structure buried in clay. Typically, the boxes in the data base were buried in sand, which generally results in less structural response than when clay backfill is used. A slab with a L/t value of approximately 5 incurred only slight damage with a support rotation of 2 degrees when the scaled range equaled 2.0 $ft/lb^{1/3}$. This slab contained single-leg stirrups, with 135-degree bends on each end, spaced at less than one-half the slab thickness. The slab that was tested in clay contained similar stirrups spaced at greater than one-half the slab thickness. As the scaled range was increased to 2.8, 4.0, and 5.0 $ft/lb^{1/3}$ for some walls, support rotations remained very small (1.5, 1.0, and

2.0 degrees, respectively).

Parameters for boxes loaded with HEST conditions are presented in Table 3.6. Although many of the HEST tests are often considered to be "highly-impulsive" by the research community, it is assumed in this discussion that they may more accurately represent tests that have a charge placed at a scaled range greater than 2.0 $ft/lb^{1/3}$. The parameter ρ varied from 0.5 to 1.2 percent for the HEST-tested roof slabs, and the boxes usually contained single-leg stirrups with a 90-degree bend on one end and a 135-degree bend on the other end. The stirrups were spaced at less than one-half the slab thickness, and the L/t values ranged from approximately 6 to 15. Generally, very little steel was ruptured in these tests. The only case in which more than 50 percent of the tension reinforcement was ruptured was for a slab with no shear reinforcement and 1.2 percent principal reinforcement. Also, the principal reinforcement in this slab was spaced at a distance greater than the slab thickness, and the slab experienced support rotations of approximately 15 degrees. When the principal reinforcement in a similar slab (ρ of 1.1 percent) was spaced at a distance less than the slab thickness, no steel was ruptured, and the slab sustained support rotations of approximately 14 degrees. In addition, a slab with single-leg stirrups (with 90- and 135-degree bends), a ρ of only 0.51 percent (principle reinforcement at a spacing less than the slab thickness), and a L/t ratio of approximately 14 achieved support rotations of approximately 16 degrees with no rupture of steel. This group (laterally-restrained boxes) of data indicated that slabs with single-leg stirrups (with 90- and 135-degree bends) and

L/d values from 6 to 15 are capable of sustaining support rotations up to approximately 30 degrees with significant damage and can achieve support rotations of approximately 25 degrees with little to no rupture of steel. Actually, this was also the case for some slabs that contained no shear reinforcement.

Laterally-Restrained Slabs

Many of the nonlaced slabs presented in Table 3.7 were tested in reaction devices for which the degree of lateral restraint cannot be determined with great confidence based on the information provided in the reports. Only two of the one-way slabs tested at scaled ranges less than 2.0 $ft/lb^{1/3}$ were definitely restrained. Although one of these was lightly reinforced (ρ equal to 0.15) with no shear reinforcement and with L/t approximately equal to 7, it sustained only "slight" damage when tested at a scaled range of 1.0 $ft/lb^{1/3}$. Unfortunately, values for support rotation or midspan deflection are not available for these slabs. Damage was described as "heavy" when the scaled range was increased to 1.25 $ft/lb^{1/3}$, L/t was decreased to 6, ρ was increased to 0.65, and looped reinforcement was used. Such variations in the data base are difficult to explain.

A considerable amount of information is available for the five two-way slabs that were laterally restrained, had L/t values of 20, and were tested at a scaled range of 2.0 $ft/lb^{1/3}$. The amounts of principal steel for these slabs (0.31, 1.0, 1.5, and 2.5 percent) included low, middle, and high values, considering the range of ρ for the data base. For values of ρ equal to 1.0 or 1.5 percent, the two-way slabs achieved support rotations of 10 to

12 degrees with no rupture of the tension steel and with "medium" damage. Even the slab having the low value of ρ equal to 0.31 percent and having no stirrups sustained a support rotation of 10.4 degrees with medium damage and no rupture of reinforcement. When ρ equaled 2.5 percent and the scaled range was 0.65 ft/lb$^{1/3}$, the support rotation was limited to 5 degrees due to the large quantity of principal reinforcement. When single-leg stirrups (180-degree bends on each end) were used, they were spaced at less than one-half the thickness of the slab.

A review of data for the laterally-restrained laced slabs tested at scaled ranges less than 2.0 ft/lb$^{1/3}$ and included in Table 3.8 provides some insight into the comparative behavior of laced and nonlaced slabs. The fact that both a laced slab and a slab with no shear reinforcement (from Table 3.7) incurred heavy damage when tested at scaled ranges of 1.5 ft/lb$^{1/3}$ and 1.25 ft/lb$^{1/3}$ respectively, somewhat questions the significance of lacing when ρ is approximately 0.65 percent. When laced slabs with a ρ of 2.7 percent were subjected to scaled ranges of 0.3 ft/lb$^{1/3}$ and 0.5 ft/lb$^{1/3}$, they experienced partial destruction and heavy damage, respectively. All parameters were the same for these two slabs except that L/t equaled 2 for the slab tested at a scaled range of 0.3 ft/lb$^{1/3}$ and L/t equaled 4 to 6 for the slabs tested at a scaled range of 0.5 ft/lb$^{1/3}$. However, a laterally-unrestrained slab (from Table 3.7) with no shear reinforcement, a ρ of 2.7, and L/t of 4 incurred only medium damage at a scaled range of 0.5 ft/lb$^{1/3}$. This indicates that the effects of the large ρ of 2.7 percent and, to some extent, the small L/t values overshadowed the effects of shear reinforcement on the response of

these slabs.

Laterally-Unrestrained Slabs

Data for laterally-unrestrained, nonlaced slabs tested at scaled ranges less than 2.0 $ft/lb^{1/3}$ are included in Table 3.7. One of these slabs contained looped shear reinforcement, had a L/t value of approximately 6, and was tested at a scaled range of 1.0 $ft/lb^{1/3}$. The damage was described as partial destruction. The rest of the slabs in the data base for this category (laterally-unrestrained slabs) contained no shear reinforcement. The damage levels ranged from slight damage to total destruction for slabs that had a L/t of approximately 8, a ρ of 0.15 percent, and were tested at scaled ranges varying from 1.7 to 1.0 $ft/lb^{1/3}$, respectively. Medium damage occurred when the scaled range equaled 1.1 $ft/lb^{1/3}$. When slabs having a L/t ratio of approximately 6 were tested at a scaled range of only 0.5 $ft/lb^{1/3}$, one with a ρ of 0.65 incurred total destruction and one with a ρ of 2.7 percent incurred heavy damage. Damage was also heavy for two unrestrained laced slabs with a L/t ratio of 6 and a ρ of 0.65 percent when tested at a scaled range of 1.0 $ft/lb^{1/3}$. It is obvious that unrestrained slabs with small amounts of tension steel are susceptible to major damage when the scaled range is less than 2.0 $ft/lb^{1/3}$.

Data for laterally-unrestrained, nonlaced slabs tested at scaled ranges greater than or equal to 2.0 $ft/lb^{1/3}$ are very limited. Four of these slabs had a L/t ratio of approximately 8 and a very low ρ of 0.15 percent. The damage levels ranged from total destruction when the scaled range equaled 2.0 $ft/lb^{1/3}$ to

slight damage when the scaled range equaled 2.6 $ft/lb^{1/3}$. Slight damage also occurred when the L/t ratio was approximately 14, ρ equaled approximately 0.4 percent, and the scaled range equaled the relatively large value of 3.5 $ft/lb^{1/3}$. None of these slabs contained any shear reinforcement.

3.6 Response Limits Based on Previous Experiments

Much of the data discussed in this chapter were taken from tests on walls or roofs of buried box structures. Most of the above-ground tests were conducted using bare (uncased) explosives, which did not produce a fragment loading and consequent degradation of the slabs.

The data from the 1960's presented in this chapter, primarily that in Table 3.3, provided the basis for the allowable response limits given in TM 5-1300. The more recent data provided that basis for the allowable response limits given in ETL 1110-9-7. From the presentation of the design criteria in Chapter 2, it is obvious that the allowable response limits given by TM 5-1300 are more conservative (allow less support rotations, particularly for slabs without lacing) than those given in ETL 1110-9-7. The greater conservatism found in TM 5-1300 is the result of a reliance on the 1960's data and the philosophy that many of the facilities designed in accordance with its criteria are utilized by civilians in peacetime operations. In contrast, the ETL relies on the more recent data that indicated that slabs with stirrups can sustain large deflections. Additionally, the ETL criteria are intended solely for the design of military facilities, where significant damage is often acceptable. Although the data review

work of this study has already impacted design criteria for military structures subjected to conventional weapons effects, the data are not adequate to significantly impact design criteria for structures to resist the effects of accidental explosions, i.e. explosives safety applications. The design of structures to resist the effects of accidental explosions is governed by TM 5-1300, which calls for the use of laced reinforcement for large deflections (support rotations greater than 8 degrees) and for close-in blast (scaled ranges less than 1.0 ft/lb$^{1/3}$). It is obvious that the safety requirements of ETL 1110-9-7 are less conservative than those of TM 5-1300 due to the military nature of structures intended to be designed in accordance with the ETL guidance. The data base on previous experiments does not include a thorough study comparing the behavior of laced and nonlaced slabs. It is rather a collection of experiments which were conducted for various purposes, thus the various design parameters are difficult to correlate between experiments. The experimental study discussed in the remainder of this paper is a first step toward a more thorough comparison of laced and nonlaced slabs.

Table 3.1 Static Slab Tests

Element	Restraint	L/t	Midspan ρ	Midspan ρ'	Support ρ	Support ρ'	f_y (ksi)	f_c' (ksi)	d (in)	t (in)	s/t	d_b (in)	Shear Reinforcem
K1-82	Rigid	8.3	0.50	0.50	0.50	0.50	90.2	6.7	2.40	2.90	0.69	0.18	Closed Ho
K2-82	Rigid	8.3	0.50	0.50	0.50	0.50	90.2	6.8	2.40	2.90	0.69	0.18	Closed Ho
K3-82	Rigid	8.3	0.50	0.50	0.50	0.50	90.2	6.7	2.40	2.90	0.69	0.18	Closed Ho
B1-83	Rigid	10.0	0.47	0.47	0.47	0.47	77.7	6.1	1.95	2.40	0.83	0.15	135-S-13
B2-83	Rigid	10.0	1.04	1.04	1.04	1.04	70.1	6.1	1.88	2.40	0.69	0.21	135-S-13
B3-83	Rigid	5.0	0.46	0.46	0.46	0.46	70.1	6.1	4.30	4.80	0.69	0.21	135-S-13
W1-83	Rigid	10.4	0.74	0.85	0.85	0.74	59.8	4.8	1.94	2.31	1.62	0.25	None
W2-83	Rigid	10.4	0.74	0.85	0.85	0.74	59.8	4.9	1.94	2.31	1.62	0.25	135-S-13
W3-83	Rigid	10.4	0.74	0.85	0.85	0.74	59.8	5.1	1.94	2.31	1.62	0.25	135-S-13
W4-83	Rigid	10.4	0.74	0.85	0.85	0.74	59.8	4.9	1.94	2.31	1.62	0.25	135-S-13
W5-83	Rigid	10.4	0.74	0.85	0.85	0.74	59.8	5.1	1.94	2.31	1.62	0.25	135-S-13
W6-83	Rigid	10.4	0.74	0.85	0.85	0.74	59.8	4.9	1.94	2.31	1.62	0.25	135-S-9
W7-83	Rigid	10.4	0.74	0.85	0.85	0.74	59.8	5.0	1.94	2.31	1.62	0.25	135-S-9
W8-83	Rigid	10.4	0.74	0.85	0.85	0.74	59.8	5.1	1.94	2.31	1.62	0.25	Double 1
W9-83	Rigid	10.4	0.75	0.86	0.86	0.75	62.4	4.7	1.94	2.31	0.76	0.18	135-S-13
W10-83	Rigid	10.4	0.75	0.86	0.86	0.75	62.4	4.9	1.94	2.31	0.76	0.18	135-S-13
W1-84	Rigid	10.4	0.74	0.85	0.85	0.74	66.0	4.5	1.94	2.31	1.62	0.25	None
W2-84	Rigid	10.4	0.79	0.79	0.79	0.79	66.0	4.5	1.81	2.31	1.62	0.25	None
W3-84	Rigid	10.4	1.14	0.40	0.40	1.14	63.5	4.5	1.81	2.31	1.62	0.38	None
W4-84	Rigid	10.4	1.14	0.40	1.19	1.14	63.5	4.5	1.81	2.31	1.62	0.178 0.30	None
W5-84	Rigid	10.4	1.14	0.40	1.19	1.14	63.5	4.5	1.81	2.31	1.62	0.178 0.30	None
W6-84	Rigid	10.4	1.58	0.00	1.58	0.00	66.0	4.5	1.81	2.31	1.62	0.25	None
W7-84	Rigid	10.4	1.13	0.45	1.13	0.45	66.0	4.3	1.81	2.31	1.62	0.25	None
W8-84	Rigid	10.4	1.13	0.45	1.13	0.45	66.0	4.3	1.81	2.31	1.62	0.25	135-S-9
W9-84	Rigid	10.4	1.13	0.45	1.13	0.45	66.0	4.0	1.81	2.31	1.62	0.25	135-S-9
W10-84	Rigid	10.4	1.13	0.45	1.13	0.45	66.0	4.0	1.81	2.31	1.62	0.25	135-S-9
W11-84	Rigid	10.4	1.13	0.45	1.13	0.45	66.0	4.2	1.81	2.31	1.62	0.25	135-S-9
W12-84	Rigid	10.4	1.13	0.45	1.13	0.45	66.0	4.2	1.81	2.31	1.62	0.25	135-S-9
W13-84	Rigid	10.4	1.13	0.45	1.13	0.45	66.0	4.2	1.81	2.31	1.62	0.25	None
W14-84	Rigid	10.4	1.02	1.02	1.02	1.02	60.3	3.6	2.40	2.90	0.69	0.25	135-S-9
W15-84	Rigid	10.4	0.79	0.45	0.79	0.45	66.0	3.6	1.81	2.31	1.62	0.25	None
G1-84	Partial	10.4	0.52	0.52	0.52	0.52	50.0	4.4	1.94	2.31	1.30	0.20	135-S-9
G2-84	Partial	10.4	0.52	0.52	0.52	0.52	50.0	4.3	1.94	2.31	1.30	0.20	135-S-9
G3-84	Partial	10.4	0.74	0.74	0.74	0.74	58.5	4.4	1.94	2.31	1.62	0.25	135-S-9
G4-84	Partial	10.4	0.74	0.74	0.74	0.74	58.5	4.3	1.94	2.31	1.62	0.25	135-S-9
G4A-84	Partial	10.4	0.74	0.74	0.74	0.74	58.5	4.2	1.94	2.31	1.62	0.25	135-S-9
G4B-84	Partial	10.4	0.74	0.74	0.74	0.74	58.5	4.2	1.94	2.31	1.62	0.25	135-S-9
G5-84	Partial	10.4	1.06	1.06	1.06	1.06	58.5	4.4	1.94	2.31	1.08	0.25	135-S-9
G6-84	Partial	10.4	1.06	1.06	1.06	1.06	58.5	4.3	1.94	2.31	1.08	0.25	135-S-9
G7-84	Partial	14.8	0.58	0.58	0.58	0.58	67.3	5.0	1.25	1.63	2.31	0.18	135-S-9
G8-84	Partial	14.8	0.58	0.58	0.58	0.58	67.3	5.0	1.25	1.63	2.31	0.18	135-S-9
G9-84	Partial	14.8	1.14	1.14	1.14	1.14	58.5	5.0	1.25	1.63	2.31	0.25	135-S-9
G9A-84	Partial	14.8	1.14	1.14	1.14	1.14	58.5	5.0	1.25	1.63	2.31	0.25	135-S-9
G10-84	Partial	14.8	1.14	1.14	1.14	1.14	58.5	5.0	1.25	1.63	2.31	0.25	135-S-9
G10A-84	Partial	14.8	1.14	1.14	1.14	1.14	58.5	5.0	1.25	1.63	2.31	0.25	135-S-9
G11-84	Partial	14.8	1.47	1.47	1.47	1.47	58.5	5.0	1.25	1.63	1.69	0.25	135-S-9
G12-84	Partial	14.8	1.47	1.47	1.47	1.47	58.5	5.0	1.25	1.63	1.69	0.25	135-S-
K48-69	Rigid	12.0	2.11	2.11	2.11	2.11	49.9	5.0	4.875	6.00	0.25	0.63	Lace
K981-69	Rigid 2-Way	24.0	0.82	0.82	0.82	0.82	49.6	3.6	2.25	3.00	2.00	0.38	Lace
K982-69	Rigid 2-Way	24.0	0.00	0.00	0.00	0.00	49.6	4.1	-	3.00	-	-	None
K983-69	Rigid 2-Way	24.0	0.82	0.82	0.82	0.82	49.6	4.1	2.25	3.00	2.00	0.38	Lace
K984-69	Rigid 2-Way	24.0	0.82	0.82	0.82	0.82	49.6	3.3	2.25	3.00	2.00	0.38	Lace
K985-69	Rigid 2-Way	15.2	0.89	0.89	0.89	0.89	47.4	3.2	3.75	4.75	1.26	0.50	Lace
K986-69	Rigid 2-Way	12.0	1.33	1.33	1.33	1.33	47.4	.6	5.00	6.00	0.50	0.50	Lace

Legend for Miscellaneous Symbols

3-H = 3-hinged mechanism
3-HM = 3-hinged mechanism membrane
DOB = depth of burial
θ_R = allowed rotational freedom at supports

ρ_s %

0.25
0.25
0.25
0.23
0.98
0.41
—
0.36
0.18
0.09
0.18
0.18
0.18
0.18
0.19
0.38
—
—
—
—
—
—
—
0.06
0.22
0.22
0.22
0.22
—
1.53
—
0.22
0.22
0.18
0.18
0.18
0.18
0.27
0.27
0.18
0.18
0.18
0.18
0.18
0.18
0.2
0.2
1.3
0.1
0.0
0.1
0.1
0.4
1.6

s/t	d_b (in)	Shear Reinforcement	ρ_s %	s_s/t	θ	I/u	Remarks
.69	0.18	Closed Hoop	0.25	0.69	0.50	1.00	3-H, test terminated at U, 100% tension steel ruptured at midspa
.69	0.18	Closed Hoop	0.25	0.69	5.70	0.90	DOB = L/2, 3-H, 100% tension steel ruptured at midspan
.69	0.18	Closed Hoop	0.25	0.69	12.90	0.32	DOB = L/2, 3-H, 100% tension and 50% comp. steel ruptured at mid
.83	0.15	135-S-135	0.23	0.81	5.20	0.77	3-H
.69	0.21	135-S-135	0.98	0.46	3.30	1.00	3-H, test terminated at U
.69	0.21	135-S-135	0.41	0.35	3.10	1.00	3-H, test terminated at U
.62	0.25	None	—	—	16.3	0.72	3-H, 86% tension steel rupture at midspan, 50% tension steel rup
.62	0.25	135-S-135	0.36	0.33	20.6	1.02	3-HM, 100% ten. & 43% comp. steel rupt. at midspan, 64% ten. rup
.62	0.25	135-S-135	0.18	0.65	14.0	0.63	3-H, 100% tension rupture at midspan, 43% tension rupture at sup
.62	0.25	135-S-135	0.09	1.30	13.1	0.55	3-H, 100% tension rupture at midspan, 29% tension rupture at sup
.62	0.25	135-S-135	0.18	0.65	15.4	0.88	Temp. steel outside, 3-H, 86% ten. rupt. @ midspan, 14% ten. rup
.62	0.25	135-S-90	0.18	0.65	14.0	0.72	3-H, 71% tension rupture at midspan, 14% tension rupture at supp
.62	0.25	135-S-90	0.18	0.65	14.5	0.85	Temp. steel outside, 3-H, 86% ten. rupt. @ midspan, 14% ten. rup
.62	0.25	Double 135	0.18	0.65	14.0	0.78	3-H, 71% tension rupture at midspan, 14% tension rupture at supp
.76	0.18	135-S-135	0.19	0.65	16.3	0.79	3-H, 100% tension rupture at midspan, 39% tension rupture at sup
.76	0.18	135-S-135	0.38	0.33	18.4	1.12	3-H, 100% tension & 57% comp rupture at midspan, 71% tension rup
.62	0.25	None	—	—	18.4	0.73	3-H, 100% tension at midspan & 7% tension at support ruptured
.62	0.25	None	—	—	19.7	0.85	3-H, 100% tension at midspan & 14% tension at support ruptured
.62	0.38	None	—	—	21.0	0.85	3-H, 71% tension at midspan & 86% tension at suuport ruptured
.62	0.178 0.30	None	—	—	20.6	0.85	2 dowels at supports. 3-HM, 43% ten. at midspan & 7% ten. at sup
.62	0.178 0.30	None	—	—	23.4	0.68	2 dowels @ supp. 3-HM, 71% ten. & 29% comp. rupt. @ midspan, 14%
.62	0.25	None	—	—	14.0	1.47	2 pairs bent. 4-H, no steel ruptured
.62	0.25	None	—	—	19.7	0.91	Alternate 2 pairs bent. 3-HM, 40% ten at midspan & 10% ten. at s
.62	0.25	135-S-90	0.06	1.30	23.4	0.93	Alternate 2 pairs bent. 3-HM, 60% ten at midspan & 20% ten. at s
.62	0.25	135-S-90	0.22	0.32	23.4	1.04	Alternate 2 pairs bent. 3-HM, 80% ten at midspan & 20% ten. at s
.62	0.25	135-S-90	0.22	0.32	23.4	0.85	Alternate 2 pairs bent. 3-HM, 60% ten & 25% comp @ mid & 45% ten
.62	0.25	135-S-90	0.22	0.32	19.3	0.74	Alternate 2 pairs bent. 3-HM, 100% ten & 50% comp @ mid & 45% te
.62	0.25	135-S-90	0.22	0.32	24.6	0.99	Alt 2 pairs bent. 3-HM, temp steel out, 50% ten at mid & 25% ten
.62	0.25	None	—	—	22.6	0.70	Alternate 2 pairs cut. 3-HM, 40% tension at midspan & 15% ten at
.69	0.25	135-S-90	1.53	0.55	22.6	0.76	3-H, 100% tension at midspan & 100% tension at support rupture
.62	0.25	None	—	—	24.2	0.69	3-H, 100% tension at midspan & 57% tension at support rupture
.30	0.20	135-S-90	0.22	1.30	17.1	0.44	θ_R = 1.82, 3-H, 100% tens & 100% comp at midspan & 88% tens at s
.30	0.20	135-S-90	0.22	1.30	19.3	0.65	θ_R = 1.56, 3-H, 100% tens & 88% comp at midspan & 88% ten at sup
.62	0.25	135-S-90	0.18	1.30	18.9	1.13	θ_R = 1.24, 3-HM, 71% tension at midspan & 29% tension at support
.62	0.25	135-S-90	0.18	1.30	18.0	1.07	θ_R = 1.50, 3-HM, 43% tension at midspan & 14% tension at support
.62	0.25	135-S-90	0.18	1.30	20.1	1.38	θ_R = 2.52, 3-HM, 57% tension at midspan rupture
.62	0.25	135-S-90	0.18	1.30	12.7	0.86	θ_R = 2.20, 3-HM, 29% tension at midspan rupture
.08	0.25	135-S-90	0.27	1.30	16.3	0.85	θ_R = 0.55, 3-HM, 30% tension at midspan & 40% tension at support
.08	0.25	135-S-90	0.27	1.30	16.3	1.33	θ_R = 2.04, 3-HM, 20% tension at midspan rupture
.31	0.18	135-S-90	0.18	1.85	17.1	0.84	θ_R = 0.61, 3-H, 86% tension & comp at midspan & 93% ten at supp
2.31	0.18	135-S-90	0.18	1.85	15.8	1.00	θ_R = 2.20, 3-H, 100% ten & 86% comp at midspan & 93% ten at sup
2.31	0.25	135-S-90	0.18	1.85	16.7	2.23	θ_R = 1.29, 3-HM, 14% tension at support rupture
2.31	0.25	135-S-90	0.18	1.85	18.0	2.24	θ_R = 0.40, 3-HM, 14% tension at support rupture
2.31	0.25	135-S-90	0.18	1.85	16.7	Large	θ_R = 2.79, pure tensile membrane, 3-HM, no steel rupture
2.31	0.25	135-S-90	0.18	1.85	11.3	Large	θ_R = 2.04, pure tensile membrane, 3-HM, 57% tension at midspan
1.69	0.25	135-S-90	0.24	1.85	14.9	2.52	θ_R = 0.76, 3-HM, no steel rupture
1.69	0.25	135-S-90	0.24	1.85	14.5	4.45	θ_R = 2.04, 3-HM, no steel rupture
0.25	0.63	Lace	1.37	0.25	9.2	1.25	Test terminated due to loading device, 3-H, no steel rupture
2.00	0.38	Lace	0.19	0.50	8.7	0.90	Loaded until rupture of steel or water seal, 3-HM
-	-	None	0.00	-	1.6	1.00	Loaded until rupture of steel or water seal, 3-H
2.00	0.38	Lace	0.19	0.50	13.3	1.23	Loaded until rupture of steel or water seal, 3-HM
2.00	0.38	Lace	0.19	0.50	12.6	0.95	Loaded until rupture of steel or water seal, 3-HM
1.26	0.50	Lace	0.42	0.42	10.8	0.87	Loaded until rupture of steel or water seal, 3-HM
0.50	0.50	Lace	1.67	0.17	1.8	0.79	Loaded until rupture of steel or water seal, 3-H

ools

s

at supports

Remarks

.00% tension steel ruptured at midspan
steel ruptured at midspan
and 50% comp. steel ruptured at midspan

t supp
t suppre at midspan, 50% tension steel rupt. at support
steel rupt. at midspan, 64% ten. rupt. at support
: midspan, 43% tension rupture at support
t supp midspan, 29% tension rupture at support
% ten. rupt. @ midspan, 14% ten. rupt. at support
t supp midspan, 14% tension rupture at support
% ten. rupt. @ midspan, 14% ten. rupt. at support
midspan, 14% tension rupture at support
t supp midspan, 39% tension rupture at support
rupture at midspan, 71% tension rupt. at support

& 7% tension at support ruptured
& 14% tension at support ruptured
: rupt. & 86% tension at suuport ruptured
43% ten. at midspan & 7% ten. at support rupt.
. @ s
:en. & 29% comp. rupt. @ midspan, 14% ten. @ supp.

. rupt ruptured
. rupt, 40% ten at midspan & 10% ten. at supp. rupture
. rupt, 60% ten at midspan & 20% ten. at supp. rupture
supp , 80% ten at midspan & 20% ten. at supp. rupture
supp , 60% ten & 25% comp @ mid & 45% ten at supp rup.
supp , 100% ten & 50% comp @ mid & 45% ten @ supp rup.
pp. ru steel out, 50% ten at mid & 25% ten at supp rup.
40% tension at midspan & 15% ten at supp. rupt.
& 100% tension at support rupture
& 57% tension at support rupture
rupt
pt 100% comp at midspan & 88% tens at supp rupt
ture 88% comp at midspan & 88% ten at supp rupt
ture at midspan & 29% tension at support rupture
at midspan & 14% tension at support rupture
at midspan rupture
ture at midspan rupture
at midspan & 40% tension at support rupture
uptur at midspan rupture
& comp at midspan & 93% ten at support rupture
ture
6% comp at midspan & 93% ten at supp rupture
at support rupture
at support rupture
re rane, 3-HM, no steel rupture
rane, 3-HM, 57% tension at midspan rupture
pture
pture

ng device, 3-H, no steel rupture

l or water seal, 3-HM
l or water seal, 3-H
l or water seal, 3-HM
l or water seal, 3-HM
l or water seal, 3-HM
l or water seal, 3-H

Table 3.2 Static Box Tests

Element	Restraint	L/t	Midspan ρ	Midspan ρ'	Support ρ	Support ρ'	f_y (ksi)	f_c' (ksi)	d (in)	t (in)	s/t	d_b (in)	Shear Reinforcement	
K1-78	4 sides	8.3	1.00	1.00	1.00	1.00	60.0	5.2	2.40	2.9	0.69	0.25	135-S-90	1.
K2-78	4 sides	8.3	1.00	1.00	1.00	1.00	72.0	6.2	2.40	2.9	0.69	0.25	135-S-90	1.
K3-79	4 sides	8.3	1.00	1.00	1.00	1.00	60.0	4.8	2.40	2.9	0.69	0.25	135-S-90	1.
K4-79	4 sides	3.3	1.85	1.85	1.85	1.85	68.0	6.1	6.40	7.3	0.69	0.75	135-S-90	1.
S1-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	6.2	1.94	2.5	1.50	0.25	D-135	0.
S2-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.2	1.94	2.5	1.50	0.25	D-135	0.
S3-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.2	1.94	2.5	1.50	0.25	D-135	0.
S4-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.6	1.94	2.5	1.50	0.25	D-135	0.
S5-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	3.5	1.94	2.5	1.50	0.25	D-135	0.
S6-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	4.5	1.94	2.5	1.50	0.25	D-135	0.

Legend for Miscellaneous Symbols

DOB = depth of burial
3-H = 3-hinged mechanism
3-HM = 3-hinged mechanism membrane

Shear nforcement	ρ_s %	s_s/t	θ	I/u	Remarks
5-S-90	1.53	0.55	5.7	1.00	DOB = L/2, Collapse U=I, 100% tension at comp. steel rupture at midspan rupt
5-S-90	1.53	0.55	3.6	1.00	DOB = L/2, 3-HM, test term at U, 80% ten. & 60% comp. @ mid, 100% ten. @ sup
5-S-90	1.53	0.55	4.8	1.00	DOB = L/5, collapse at U=I, 100% tension & comp. steel rupture @ midspan rup
5-S-90	1.10	0.69	9.9	0.91	DOB = L/5, shear failure, no steel ruptured
D-135	0.18	0.60	1.7	1.00	DOB = 4L/11, collapse at U=I, interior support failed
D-135	0.18	0.60	16.9	0.47	3-H, 100% tension at midspan and support rupture
D-135	0.18	0.60	15.6	0.46	DOB = 4L/11, 3-H, 100% tension at midspan and support rupture
D-135	0.18	0.60	7.9	0.72	DOB = 4L/11, 3-H, 100% tension at midspan and support rupture
D-135	0.18	0.60	15.3	0.55	DOB = 4L/11, 3-H, 100% tension at midspan and support rupture
D-135	0.18	0.60	15.3	0.98	DOB = 4L/11, 3-H, 100% tension at midspan and support rupture

Table 3.3 Dynamic Slab Tests

Legend for Restraint

H-1 = Slab in horizontal position and supported on horizontal wood blocks
H-2 = Slab in horizontal position and supported on vertical steel blocks
-3 = Slab in horizontal position and supported on horizontal steel blocks
-1 = Slab in vertical position, bolted in modified "new structure" with lateral restraining plates
V-2 = Slab in vertical position, located in steel cubicle and supported by steel frame
-3 = Slab in vertical position and supported by steel tunnel
-4 = Slab in vertical position and supported in steel tunnel or in "new structure"

Legend for Reinforcement Types

RB = commercial reinforcing bar
CWF = commercial welded wire fabric
CWW = commercial welded wire

Element	Restraint	L/t	Midspan ρ	Midspan ρ'	Support ρ	Support ρ'	f_y (ksi)	f_c' (ksi)	d (in)	t (in)	s/t	d_b (in)
FS-1-63-1	H-1	8	0.15	0.15	0.15	0.15		4.0		12	1.0	0.50
FS-1-63-2	H-1	8	0.15	0.15	0.15	0.15		4.0		12	1.0	0.50
FS-1-63-3	H-1	8	0.15	0.15	0.15	0.15		4.0		12	1.0	0.50
FS-1-63-4	H-1	8	0.15	0.15	0.15	0.15		4.0		12	1.0	0.50
FS-1-63-5	H-1	8	0.15	0.15	0.15	0.15		4.0		12	1.0	0.50
1/3-1-63-1	H-1	8	0.15	0.15	0.15	0.15		6.0		4	1.0	0.16
1/3-1-63-2	H-1	8	0.15	0.15	0.15	0.15		6.0		4	1.0	0.16
1/3-1-63-3	H-1	8	0.15	0.15	0.15	0.15		6.0		4	1.0	0.16
1/3-1-63-4	H-1	8	0.15	0.15	0.15	0.15		6.0		4	1.0	0.16
1/3-1-63-5	H-1	8	0.15	0.15	0.15	0.15		6.0		4	1.0	0.16
1/3-1-63-6	H-1	8	0.15	0.15	0.15	0.15		6.0		4	1.0	0.16
FS-1-64-1	H-1	8	0.15	0.15	0.15	0.15		>=2.5		12	1.0	0.50
FS-1-64-2	V-1	8	0.15	0.15	0.15	0.15		>=2.5		12	1.0	0.50
FS-1-64-3	V-2	8	0.15	0.15	0.15	0.15		>=2.5		12	1.0	0.50
1/3-1-64-1	V-2	8	0.15	0.15	0.15	0.15		>=5.0		4	1.0	0.16
1/3-1-64-2	V-1	8	0.15	0.15	0.15	0.15		>=5.0		4	1.0	0.16
1/3-1-64-3	H-1	8	0.15	0.15	0.15	0.15		>=5.0		4	1.0	0.16
CAM-1-64-1	H-2	8	0.15	0.15	0.15	0.15		5.1		4	1.0	0.16
CAM-1-64-1	H-3	8	0.15	0.15	0.15	0.15		5.1		4	1.0	0.16
CAM-1-64-1	V-3	8	0.15	0.15	0.15	0.15		5.1		4	1.0	0.16
BAL-64-1	V-3	8	1.30	1.30	1.30	1.30				4	0.6	0.38
BAL-64-2	V-3	8	1.30	1.30	1.30	1.30				4	0.6	0.38
1/3-2-64-1	V-3	8	0.15	0.15	0.15	0.15		4.9		4	1.0	0.16
1/3-2-64-2	V-3	8	0.15	0.15	0.15	0.15		4.9		4	1.0	0.16
1/3-2-64-3	V-3	8	0.15	0.15	0.15	0.15		4.9		4	1.0	0.16
1/3-2-64-4	V-3	8	0.15	0.15	0.15	0.15		4.9		4	1.0	0.16
1/3-2-64-5	V-3	8	0.15	0.15	0.15	0.15		4.9		4	1.0	0.16
1/3-S1-64-1	V-3	14	0.40	0.40	0.40	0.40		>=6.0		4	0.5	0.19
1/3-S1-64-2	V-3	14	0.40	0.40	0.40	0.40		>=6.0		4	0.5	0.19
1/3-S1-64-3	V-3	14	0.40	0.40	0.40	0.40		>=6.0		4	0.5	0.19
1/3-S1-64-4	V-3	14	0.40	0.40	0.40	0.40		>=6.0		4	0.5	0.19
1/3-S2-65-1	V-4	6	0.44	0.44	0.44	0.44				4		
1/3-S2-65-2	V-4	6	0.44	0.44	0.44	0.44				4		
1/3-S3-65-1	V-4	6	0.65	0.65	0.65	0.65				4		
1/3-S3-65-2	V-4	6	0.65	0.65	0.65	0.65				4		
1/3-S4-65-1	V-4	6	0.65	1.40	1.40	0.65				4		
1/3-S4-65-2	V-4	6	0.65	1.40	1.40	0.65				4		
1/3-S4-65-3	V-4	6	0.65	1.40	1.40	0.65				4		
1/3-S4-65-4	V-4	6	0.65	1.40	1.40	0.65				4		
1/3-S4-65-5	V-4	6	0.65	1.40	1.40	0.65				4		
1/3-S4-65-6	V-4	6	0.65	1.40	1.40	0.65				4		
1/3-S4-65-7	V-4	6	0.65	1.40	1.40	0.65				4		
1/3-S4-65-8	V-4	6	0.65	1.40	1.40	0.65				4		
1/3-S4-65-9	V-4	6	0.65	1.40	1.40	0.65				4		
1/3-S4-65-10	V-4	6	0.65	1.40	1.40	0.65				4		
1/3-S4-65-11	V-4	6	0.65	1.40	1.40	0.65				4		
1/3-S4-65-12	V-4	6	0.65	1.40	1.40	0.65				4		
1/3-S4-65-13	V-4	6	0.65	1.40	1.40	0.65				4		

1

for Reinforcement Types

mercial reinforcing bar
mercial welded wire fabric
mercial welded wire

Shear forcement	t (in)	s/t	d_b (in)	Shear Reinforcement	ρ_s %		Reinf. Type	Z (ft/lb$^{1/3}$)	Remarks
None									
None	12	1.0	0.50	None	--	--	RB	2.62	Surface pitted. Slight damage
None	12	1.0	0.50	None	--	--	RB	1.68	Surface pitted, hairline cracks. Slight da
None	12	1.0	0.50	None	--	--	RB	1.08	Partial surface crushing, large cracks. Me
None	12	1.0	0.50	None	--	--	RB	1.04	Partial crushing, small rubble. Complete f
None	12	1.0	0.50	None	--	--	RB	1.67	Partial crushing, small rubble. Complete f
None	4	1.0	0.16	None	--	--	CWW	1.02	Broken into two sections. Failure
None	4	1.0	0.16	None	--	--	CWW	1.72	Hairline cracks. Slight damage
None	4	1.0	0.16	None	--	--	CWW	1.01	Reduced to small rubble. Complete failure
None	4	1.0	0.16	None	--	--	CWW	1.72	Several large sections and small rubble. C
None	4	1.0	0.16	None	--	--	CWW	0.99	Broken into two sections. Failure
	4	1.0	0.16	None	--	--	CWW	2.59	Partial crushing, small rubble. Failure
None									
None	12	1.0	0.50	None	--	--	RB	2.57	Partial crushing, small rubble. Complete f
None	12	1.0	0.50	None	--	--	RB	1.01	Medium cracks. Slab displaced 20-30 ft. S
None	12	1.0	0.50	None	--	--	RB	1.01	Pitted and cracked. Slab and support displ
None	4	1.0	0.16	None	--	--	CWF	1.02	Broken into two sections. Failure
None	4	1.0	0.16	None	--	--	CWF	1.02	Broken into two sections. Failure
	4	1.0	0.16	None	--	--	CWF	2.57	No damage, slight
None									
None	4	1.0	0.16	None	--	--	CWF	0.49	Reduced to small rubble. Complete failure.
None	4	1.0	0.16	None	--	--	CWF	0.46	Reduced to small rubble. Complete failure.
	4	1.0	0.16	None	--	--	CWF	0.47	Reduced to small rubble. Complete failure.
None									
None	4	0.6	0.38	None	--	--	RB	2.47	Pitted, large cracks with rubble. Heavy da
	4	0.6	0.38	None	--	--	RB	0.50	Reduced to small rubble. Complete failure.
None									
None	4	1.0	0.16	None	--	--	CWW	1.99	Broken into two sections. Failure
None	4	1.0	0.16	None	--	--	CWW	1.51	Broken into two sections with supplementary
None	4	1.0	0.16	None	--	--	CWW	0.50	Large and small rubble. Complete failure
None	4	1.0	0.16	None	--	--	CWW	3.51	No wood blocks. Temperature steel failed.
	4	1.0	0.16	None	--	--	CWW	2.52	Large cracks. Medium damage
lone									
lone	4	0.5	0.19	None	--	--	CWW	0.50	Center reduced to small rubble. Complete
lone	4	0.5	0.19	None	--	--	CWW	1.51	Tension steel failed. Large deflection.
lone	4	0.5	0.19	None	--	--	CWW	3.51	Hairline cracks. Slight damage.
	4	0.5	0.19	None	--	--	CWW	2.52	Hairline cracks. Slight damage.
lone									
lone	4			None	--	--	wire	0.50	Total destruction. Disintegration at cent
lone	4			None	--	--	wire	1.25	Heavy damage. No steel failure. Several
lone	4			None	--	--	RB	0.50	Heavy damage. No steel failure. Bent int
lone	4			None	--	--	RB	1.25	Heavy damage. No steel failure. Several
lone	4			None	--	--	RB	0.50	Heavy damage. No steel failure. Not quit
lone	4			None	--	--	RB	0.50	Total destruction. Disintegration of cent
lone	4			None	--	--	RB	1.25	Total destruction. Disintegration of conc
lone	4			None	--	--	RB	1.00	Total destruction. Disintegration of conc
one	4			None	--	--	RB	0.50	Total destruction. Disintegration of conc
one	4			None	--	--	RB	1.60	Partial destruction. Shear failure of cor
one	4			None	--	--	RB	0.55	Total destruction. Disintegration of conc
one	4			None	--	--	RB	0.80	Total destruction. Disintegration of conc
one	4			None	--	--	RB	1.25	Total destruction. Disintegration of conc
one	4			None	--	--	RB	0.80	Total destruction. Disintegration of conc
one	4			None	--	--	RB	0.80	Total destruction. Disintegration of conc
one	4			None	--	--	RB	1.25	Partial destruction. Shear failure.
	4			None	--	--	RB	1.00	Total destruction. Disintegration of conc

2

Remarks

	:ght damage
, damage	:line cracks. Slight damage
re	ahing, large cracks. Medium damage
re	aall rubble. Complete failure
	aall rubble. Complete failure
	:ions. Failure
,ete failu:	:ight damage
	ble. Complete failure
	ns and small rubble. Complete failure
	:ions. Failure
	aall rubble. Failure
ure	
it damage	aall rubble. Complete failure
1. Medium	; displaced 20-30 ft. Slight damage
	Slab and support displaced. Medium damage
	.ions. Failure
	.ions. Failure
1/3 scale)	
1/3 scale)	ble. Complete failure. (1/3 scale)
1/3 scale)	ble. Complete failure. (1/3 scale)
	ble. Complete failure. (1/3 scale)
e. (1/3 s	
1/3 scale)	with rubble. Heavy damage. (1/3 scale)
	ble. Complete failure. (1/3 scale)
acks. Fai	ions. Failure
	.ions with supplementary cracks. Failure
all defl.	.le. Complete failure
	perature steel failed. Small defl. Heavy damage.
	m damage
ure	
y Damage	aall rubble. Complete failure
	.. Large deflection. Heavy Damage
	ight damage.
	ight damage.
Diag fail	
r cracks.	Disintegration at center. Diag failure.
o sections	eel failure. Several major cracks. Spalling
r cracks.	:eel failure. Bent into two sections.
nt into tw	:eel failure. Several major cracks. Spalling.
Diag. fai	:eel failure. Not quite bent into two sections.
.	Disintegration of center. Diag. failure. (+) steel rupt.
.	Disintegration of concrete.
.	Disintegration of concrete.
e.	Disintegration of concrete.
and steel	Shear failure of concrete.
and steel	Disintegration of concrete and steel.
.	Disintegration of concrete and steel.
and steel	Disintegration of concrete.
.	Disintegration of concrete and steel.
	Disintegration of concrete.
.	Shear failure.
	Disintegration of concrete.

3

Table 3.3 Dynamic Slab Tests (continued)

Legend for Restraint

V-4 = Slab in vertical position and supported in steel tunnel or in "new structure"
V-5 = Slab in vertical position in steel tunnel or "new structure" with bending restraint plates, but not laterally restrained
V-6 = Slab in vertical position in "new structure," bolted with one row of bolts at each support
V-3 = Slab in vertical position and supported by tunnel
H-4 = Slab in horizontal position in steel tunnel

Legend for Reinforcement Type

RB = commercial reinforcing bar

Element	Restraint	L/t	Midspan ρ	Midspan ρ'	Support ρ	Support ρ'	f_y (ksi)	f_c' (ksi)	d (in)	t (in)	s/t	d_b (in)	Rein
1/3-S6-65-1	V-4	6	0.65	1.40	1.40	0.65				4			
1/3-S6-65-2	V-4	6	0.65	1.40	1.40	0.65				4			
1/3-S7-65-1	V-4	2	0.15	0.15	0.15	0.15				12			
1/3-S8-65-1	V-4	2	0.69	1.33	1.33	0.69				12			
1/3-S9-65-1	V-4	1.85	0.15	0.15	0.15	0.15				13			
1/3-S10-65-1	V-5	6	0.65	1.40	1.40	0.65				4			
1/3-S10-65-2	V-5	6	0.65	1.40	1.40	0.65				4			
1/3-S11-65-1	V-5	6	0.65	0.65	0.65	0.65				4			
1/3-S11-65-2	V-5	6	0.65	0.65	0.65	0.65				4			
1/3-S12-65-1	V-5	6	0.65	0.65	0.65	0.65				4			
1/3-S13-65-1	V-5	6	2.70	2.70	2.70	2.70				4			
1/3-S13-65-2	V-5	6	2.70	2.70	2.70	2.70				4			
1/3-S14-65-1	V-5	4	2.70	2.70	2.70	2.70				6			
1/3-S15-65-1	V-5	6	0.75	0.75	0.75	0.75				4			
1/3-S11-66-1	V-6	6	0.65	0.65	0.65	0.65				4			
1/3-S11-66-2	V-6	6	0.65	0.65	0.65	0.65				4			
1/3-S11-66-3	V-6	6	0.65	0.65	0.65	0.65				4			
1/3-S11-66-4	V-6	6	0.65	0.65	0.65	0.65	Nylon	fiber	added	4			
1/3-S11-66-5	V-6	6	0.65	0.65	0.65	0.65		low		4			
1/3-S11-66-6	V-6	6	0.65	0.65	0.65	0.65	Cut steel	wire	added	4			
1/3-S12-66-1	V-6	6	0.65	0.65	0.65	0.65				4			
1/3-S13-66-1	V-6	6	2.70	2.70	2.70	2.70	Cut steel	wire	added	4			
1/3-S13-66-2	V-6	6	2.70	2.70	2.70	2.70	Nylon	fiber	added	4			
1/3-S13-66-3	V-6	6	2.70	2.70	2.70	2.70	Cut steel	wire	added	4			
1/3-S13-66-4	V-6	6	2.70	2.70	2.70	2.70		low		4			
1/3-S14-66-1	V-6	4	2.70	2.70	2.70	2.70				6			
1/3-S16-66-1	V-6	2	2.70	2.70	2.70	2.70				12			
1/8-1-66-1	H-4	6	0.15	0.15	0.15	0.15				1.50			
1/8-S1-66-1	V-3	6	0.65	0.65	0.65	0.65				1.50			
1/8-S2-66-1	V-3	6	0.65	1.40	1.40	0.65				1.50			
1/8-S2-66-2	V-3	6	0.65	1.40	1.40	0.65				1.50			
1/8-S2-66-3	V-3	6	0.65	1.40	1.40	0.65				1.50			
1/8-S2-66-4	V-3	6	0.65	1.40	1.40	0.65				1.50			
1/8-S2-66-5	H-4	6	0.65	1.40	1.40	0.65				1.50			

1

ieinforcement Type

ial reinforcing bar

<u>Legend for Miscellaneous Symbols</u>

MD = Medium Damage - less than incipient failure
HD = Heavy Damage - at or around incipient failure condition, scabbing and/or crushed concrete between reinforcement
PD = Partial Destruction - slab broken-up but remaining in one piece
TD = Total Destruction - slab broken-up completely, producing flying fragments

r ement	t (in)	s/t	d_b (in)	Shear Reinforcement	ρ_s %		Reinf. Type	Z (ft/lb$^{1/3}$)	Remarks
e	4			Lace	0.40		RB	0.50	Partial destruction. (+) steel failed. Concrete c
e	4			Lace	0.40		RB	1.25	Medium damage. Steel intact. Minor spalling.
e	12			None	—	—	RB	0.50	Total destruction. Diag. failure. Steel failure.
e	12			Lace	0.53		RB	0.50	Heavy damage. Steel int. Several cracks tot dept
e	13			Lace	0.53		RB	0.40	Reinforcement intact. Heavy spalling.
e	4			Lace	0.40		RB	1.00	Medium damage. Steel intact. Some spalling of bo
e	4			Lace	0.40		RB	0.80	Medium damage. Steel intact. Some spalling of bo
e	4			Lace	0.15		RB	0.80	Partial destruction. Tension steel failed. Compl
e	4			Lace	0.15		RB	1.00	Heavy damage. Failure of tension steel @ one supp 75% for both surfaces.
p	4			Loop	0.30		RB	1.00	Partial destruction. All tension steel failed. S
e	4			Lace	1.20		RB	0.50	Hvy dam. No steel failure. Large deflection. Co
e	4			Lacc	1.20		RB	0.50	Hvy dam. Tension steel failed @ both supp & cen.
e	6			None	—	—	RB	0.50	Medium damage. All steel intact. Complete spalli
e	[illegible]			None	—	—	RB	0.42	Total destruction. Most steel failed.
e	[illegible]			Lace	0.15		RB	1.00	Heavy damage. All steel intact. Heavy scabbing h Concrete crushed on bottom.
e	[illegible]			Lace	0.15		RB	1.00	Medium damage. Wood support blocks. Steel intact Delta max = 2.5"
e	4			Lace	0.15		RB	1.00	Heavy damage Incipient Tension steel failed at cen Concrete crushed between steel.
e	4			Lace	0.15		RB	1.00	Tension steel failed both supports and center. Co Partial destruction.
e	4			Lace	0.15		RB	1.00	Total destruction. All steel failed. Concrete di between steel.
e	4			Lace	0.15		RB	1.25	Partial destruction. Tension steel failed. Parti acceptor surface.
p	4			Loop	0.30		RB	1.25	No reinforcement failure. Major cracks at corner Spalling at center. MD.
e	4			Lace	1.20		RB	0.75	Complete surface spalling. Lower 5" of concrete d
e	4			Lace	1.20		RB	0.75	No spalling. Slight cracking on donor side. All Δmax = 2-7/8". MD.
e	4			Lace	1.20		RB	0.75	Center of panel crushed. Major cracks at support in middle (acc). All steel intact. Δmax = 3-1/8'
e	4			Lace	1.20		RB	0.75	All flexural steel intact. Several ties failed. spalling (donor and acceptor). HD.
e	6			Lace	1.20		RB	0.50	No steel failure. Complete spalling on both side Scabbing at midspan. HD
e	12			Lace	1.20		RB	0.30	All flexural steel intact. Ties fail at bonds. Broken cone fell out. PD.
	1.50			None	—			0.46	PD. Broke through at center. Large and small fr Acceptor face cracked.
	1.50			Lace	0.15			0.50	Donor spalled and cracked at supports. Acc. spal Positive steel failure at center.
	1.50			None	—			0.50	TD. Positive steel failed at supports. Center p completely destroyed.
	1.50			None	—			0.50	TD. Positive steel failed at center. Center por completely destroyed.
	1.50			None	—			0.50	TD. Positive steel failed at supports. Center p completely destroyed.
	1.50			None	—			0.50	TD. Positive steel failed at center. Center por completely destroyed.
	1.50			None	—			0.50	TD. Center portion completely destroyed. Steel (donor) and center (acceptor).

2

llaneous Symbols

than incipient failure
around incipient failure
nd/or crushed concrete

slab broken-up but remaining

lab broken-up completely,
ments

Remarks

ion. (+) steel failed. Concrete crushed at center
Steel intact. Minor spalling.
n. Diag. failure. Steel failure. Center shattered.
teel int. Several cracks tot depth. Deep spalling.
tact. Heavy spalling.
Steel intact. Some spalling of both surfaces.
Steel intact. Some spalling of both surfaces.
ion. Tension steel failed. Complete spalling.
ailure of tension steel @ one support and center. Spalling
faces.
ion. All tension steel failed. Shear failure in concrete.
el failure. Large deflection. Complete spalling both side.
n steel failed @ both supp & cen. 11" deflection at center.
All steel intact. Complete spalling of acc. surface.
n. Most steel failed.

ll steel intact. Heavy scabbing both faces.
on bottom.
Wood support blocks. Steel intact. Acc. spalling @ center.

ipient Tension steel failed at center. Heavy spalling.
between steel.
iled both supports and center. Concrete undamaged.
ion.
n. All steel failed. Concrete dislocated from

tion. Tension steel failed. Partial spalling on-
e.
t failure. Major cracks at corners.
ter. MD.
spalling. Lower 5" of concrete disintegrated.

ight cracking on donor side. All steel intact.
D.
crushed. Major cracks at support (donor). Spalling
. All steel intact. Δmax = 3-1/8". MD.
el intact. Several ties failed. Complete
and acceptor). MD.
. Complete spalling on both sides.
span. MD
el intact. Ties fail at bonds. Slab disintegration.
out. PD.

gh at center. Large and small fragments.
racked.
d cracked at supports. Acc. spalling and cracking.
failure at center.
teel failed at supports. Center portion of slab
royed.
teel failed at center. Center portion of slab
royed.
teel failed at supports. Center portion of slab
royed.
teel failed at center. Center portion of slab
royed.
rtion completely destroyed. Steel broke at supports
ter (acceptor).

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Table 3.3 Dynamic Slab Tests (continued)

Legend for Reinforcement Types

RS = commercial reinforcing bar

Legend for Restraint

V-3 = Slab in vertical position and supported by tunnel
V-6 = Slab in vertical position in "new structure", bolted with one row of bolts at each support
V-7 = Slab bolted in modified "new structure" with lateral restraining plates.
H-4 = Slab in horizontal position in steel tunnel

Element	Restraint	L/t	Midspan ρ	Midspan ρ'	Support ρ	Support ρ'	f_y (ksi)	f_c' (ksi)	d (in)	t (in)	s/t	d_b (in)	She Reinfo
1/8-S3-66-1	V-3	6	0.65	1.40	1.40	0.65				1.50			La
1/8-S3-66-2	V-3	6	0.65	1.40	1.40	0.65				1.50			La
1/8-S4-66-1	V-3	6	0.65	1.40	1.40	0.65				1.50			La
1/8-S4-66-2	V-3	6	0.65	1.40	1.40	0.65				1.50			La
1/8-S4-66-3	V-3	6	0.65	1.40	1.40	0.65				1.50			La
1/8-S5-66-1	V-3	4	2.70	2.70	2.70	2.70				2.25			La
1/8-S5-66-2	V-3	4	2.70	2.70	2.70	2.70				2.25			La
1/8-S5-66-3	V-3	4	2.70	2.70	2.70	2.70				2.25			La
1/3-S11-67-1	V-7	6	0.65	0.65	0.65	0.65		low		4.00			La
1/3-S11-67-2	V-7	6	0.65	0.65	0.65	0.65	Cut steel	wire	added	4.00			La
1/3-S11-67-3	V-7	6	0.65	0.65	0.65	0.65	Cut steel	wire	added	4.00			La
1/3-S11-67-4	V-7	6	0.65	0.65	0.65	0.65	Nylon	fiber		4.00			La
1/3-S11-67-5	V-7	6	0.65	0.65	0.65	0.65	Nylon	fiber		4.00			La
1/3-S13-67-1	V-7	6	2.70	2.70	2.70	2.70				4.00			La
1/3-S13-67-2	V-7	6	2.70	2.70	2.70	2.70	Cut steel	wire	added	4.00			La
1/3-S13-67-3	V-7	6	2.70	2.70	2.70	2.70	Cut steel	wire	added	4.00			La
1/3-S13-67-4	V-7	6	2.70	2.70	2.70	2.70		low		4.00			La
1/3-S13-67-5	V-7	6	2.70	2.70	2.70	2.70		low		4.00			La
1/3-S14-67-1	V-7	6	2.70	2.70	2.70	2.70				6.00			La
1/3-S14-67-2	V-7	4	2.70	2.70	2.70	2.70				6			La
1/3-S16-67-1	V-7	2	2.70	2.70	2.70	2.70				12			La
1/3-S16-67-2	V-7	2	2.70	2.70	2.70	2.70				12			La
1/3-S17-67-1	V-7	2	2.70	2.70	2.70	2.70				4			La

for Reinforcement Types

mercial reinforcing bar

Legend for Miscellaneous Symbols

MD = Medium Damage - less than incipient failure
HD = Heavy Damage - at or around incipient failure condition, scabbing and/or crushed concrete between reinforcement
PD = Partial Destruction - slab broken-up but remaining in one piece
TD = Total Destruction - slab broken-up completely, producing flying fragments

ent	ρ	t (in)	s/t	d_b (in)	Shear Reinforcement	ρ_s %	Reinf. Type	Z ($ft/lb^{1/3}$)	Remarks
	0.	1.50			Lace	0.40		0.50	PD. Donor badly cracked and broken through. Pos steel failed.
	0.	1.50			Lace	0.40		0.80	MD. No steel failed. Donor cracking and spal Acceptor cracking and spalling. Δ = 1/4".
	0.	1.50			Lace	0.40		0.50	MD. Donor slightly spalled. Acceptor deeply No steel failed. No deflection.
	0.	1.50			Lace	0.40		0.40	MD. Donor spalled and cracked. Acceptor dee Δmax = 3/16"
	0.	1.50			Lace	0.40		0.40	MD. Donor spalled and cracked. Acceptor dee Δmax = 3/16"
	1.	2.25			Lace	1.20		0.50	MD. Donor spalled and cracked. Acceptor dee Small deflection
	1.	2.25			Lace	1.20		0.50	MD. Donor spalled and cracked. Acceptor dee Δmax = 3/4"
	1.	2.25			Lace	1.20		0.50	MD. Donor spalled and cracked. Acceptor dee Δmax = 3/4"
	0.	4.00			Lace	0.15		1.50	HD. Donor (complete spall, one lacing failed all flexural steel intact). Acceptor (complet all steel intact, Δ = 6", horizontal movement
	0. d	4.00			Lace	0.15		1.50	HD. Donor (no spall, flexural steel failed in Δmax = 4"). Acceptor (no spall, flex steel fa center, just beyond incipient failure).
	0. d	4.00			Lace	0.15		1.50	HD. Donor (no spall, flexural steel failed in Δmax = 4"). Acceptor (no spall, flex steel fa center, just beyond incipient failure).
	0.	4.00			Lace	0.15		1.65	MD. No steel fail, no spall, crack at both su donor center, Δ = 2.9"
	0.	4.00			Lace	0.15		1.65	MD. No steel fail, no spall, crack at both su donor center, Δ = 2.9"
	1.	4.00			Lace	1.20		1.00	HD. Donor (complete spall exc 8" vertical st Δmax = 5" horizontal movement). Acc (complet except 6" wide vertical strip at rt. support,
	1. d	4.00			Lace	1.20		0.90	HD. Donor (no spall, slight crush, all steel spall, all steel intact), Δmax = 3-1/4".
	1. d	4.00			Lace	1.20		0.90	HD. Donor (no spall, slight crushing, all st (No spall, all steel intact), Δmax = 3-1/4".
	1.	4.00			Lace	1.20		1.00	HD. Complete spalling, all steel intact, Δma horizontal movement.
	1.	4.00			Lace	1.20		1.00	HD. Complete spalling, all steel intact, Δma horizontal movement.
	1.	6.00			Lace	1.20		0.50	HD. Donor (complete spall, 2 laces broke alo chopped at bottom. Acceptor (comlete spall, Δmax = 3").
	1.	6			Lace	1.20		0.50	HD. Donor (nearly complete spall, one lace f steel intact. Acceptor (complete spall, 3 la
	1.	12			Lace	1,20		0.30	HD. Complete spall both sides, lace fail at no flexural steel failure, Δ = 2".
	1.	12			Lace	1.20		0.35	HD. Complete spall both sides, one flex and acceptor center, one lace failed at donor cen
	1.	4			Lace	1.20		0.90	HD. Donor (complete spall except 6" vert str Acc (comp spalling, all steel intact), Δmax =

2

broken

through. Acc. broken through.

at supp

ng and spalling at supports.

led. = 1/4".

tor deeply spalled.

palled.

ceptor deeply spalled.

palled.

ceptor deeply spalled.

palled.

ceptor deeply spalled.

palled.

ceptor deeply spalled.

palled.

ceptor deeply spalled.

support,

alling, ing failed at support,

slab). or (complete spalling,

ver 1/2 l movement of slab).

l at l failed in lower 1/2 at rt. supp.

ex steel failed at

re).

ver 1/2

l at l failed in lower 1/2 at rt. supp.

ex steel failed at

re).

rts comp

at both supports comp crush

rts comp

at both supports comp crush

all ste

lling ertical strip, all steel intact,

steel i (complete spalling

. support, all steel intact).

ct). A

all steel intact). Acc (no

intact). 1/4".

ng, all steel intact). Acc.

-1/2". = 3-1/4".

ntact, Δmax = 3-1/2".

-1/2".

ntact, Δmax = 3-1/2".

eft supp

steel in broke along left support, concrete

te spall, all steel intact

l at rt.

fail at one lace failed at rt. supp. flex

1/2 of pall, 3 laced fail at center.

s fail at upper 1/2 of slab,

es fail

Δ = 2.4 flex and 2 laces failed at

ll rein donor center, Δ = 2.4"

horz mo vert strip, all reinfor. intact).

t), Δmax = 5", horz mor.).

3

Table 3.3 Dynamic Slab Tests (continued)

Legend for Restraint

V-7 = Slab bolted in modified "new structure" with lateral restraining plates

Legend for Reinforcement Types

RS = commercial reinforcement bar

Element	Restraint	L/t	Midspan ρ	Midspan ρ'	Support ρ	Support ρ'	f_y (ksi)	f_c' (ksi)	d (in)	t (in)	s/t	d_b (in)	Shear Reinforcement
1/3-S17-67-2	V-7	6	2.70	2.70	2.70	2.70				4			Lace
1/3-S18-67-1	V-7	4	2.70	2.70	2.70	2.70				6			Lace
1/3-S18-67-2	V-7	4	2.70	2.70	2.70	2.70				6			Lace
1/3-S18-67-3	V-7	4	2.70	2.70	2.70	2.70				6			Lace
T-1-88	4 sides 2-way slab	20	1.00	1.00	1.00	1.00	74.5	4.0		4.5	0.33	0.25	single 180
T-2-88	4 sides 2-way slab	20	1.00	1.00	1.00	1.00	74.5	4.0		4.5	0.33	0.25	Lace
T-3-88	4 sides 2-way slab	20	1.50	1.50	1.50	1.50	74.5	4.0		4.5	0.56	0.25	single 180
T-4-88	4 sides 2-way slab	20	1.00	1.00	1.00	1.00	74.5	4.0		4.5	0.33	0.23	single 180
T-5-88	4 sides 2-way slab	15	0.31	0.31	0.31	0.31	74.5	4.0		6.0	0.67	0.25	None
T-6-88	4 sides 2-way slab	20	2.50	2.50	2.50	2.50	66.0	4.0		4.5	0.33	0.38	single 180
K4D1-69	Rigid	12	2.11	2.11	2.11	2.11	49.9	5.7	4.88	6.0	0.50	0.63	Lace
K4D2-69	Rigid	12	2.11	2.11	2.11	2.11	49.9	5.4	4.88	6.0	0.50	0.63	Lace
K4D3-69	Rigid	12	2.11	2.11	2.11	2.11	49.9	5.5	4.88	6.0	0.50	0.63	Lace
K9D1-69	Rigid 2-way slab	24	0.82	0.82	0.82	0.82	49.6	3.8	2.25	3.0	2.00	0.38	Lace
K9D2-69	2-way slab Rigid	15.2	0.89	0.89	0.89	0.89	47.4	3.3	3.75	4.75	1.26	0.50	Lace
K9D3-69	2-way slab Rigid	15.2	0.89	0.89	0.89	0.89	47.4	3.6	3.75	4.75	1.26	0.50	Lace

Legend for Miscellaneous Symbols

d = Slab loaded in chamber with explosives distributed in firing tubes
3-H = 3-hinged mechanism
P_{so} = Peak surface overpressure
HD = High Damage - at or around incipient failure condition, scabbing and/or crushed concrete between reinforcement

%		Shear Reinforcement	ρ_s %		Reinf. Type	Z (ft/lb$^{1/3}$)	Remarks
20		Lace	1.20			1.00	HD. Donor (near complete spall, all steel intact). Acc. (near complete spall all steel intact, ΔMax = 5", horz. mov.).
.20		Lace	1.20			0.50	HD. Complete spalling, no steel failed, Δ = 1.7"
.20		Lace	1.20			0.50	HD. Complete spalling, no steel failed, Δ = 1.7"
.20		Lace	1.20			0.50	HD. Complete spalling, all flexural steel intact, one lace failed a[illegible] right support, Δ = 3.5"
.45	0.[illegible]	single 180	0.45	0.33	RB	0.69	θ = 10.1, no steel failed
.22	0.[illegible]	Lace	0.22	0.67	RB	0.74	θ = 9.3, no steel failed
.48	0.[illegible]	single 180	0.48	0.56	RB	0.65	θ = 10.5, no steel failed
.47	0.[illegible]	single 180	0.47	0.67	RB	0.69	θ = 12.2, no steel failed
—	—	None	—	—	RB	1.10	θ = 10.4, no steel failed; 2.5" long shear crack at 1 support
.89 .45	0.[illegible]	single 180	0.89 0.45	0.33	RB	0.65	θ = 4.0, no steel failed
.37	0.[illegible]	Lace	1,37	0.50	RB	d	θ = 5.2 on 4th loading, P_{so} = 106 psi, 3-H, No steel rupture
.37	0.[illegible]	Lace	1.37	0.50	RB	d	θ = 9.2 on 2nd loading, P_{so} = 206 psi, 3-H, No steel rupture
.37	0.[illegible]	Lace	1.37	0.50	RB	d	θ = 7.6 on 2nd loading, P_{so} = 229 psi, 3-H, No steel rupture
.19	0.[illegible]	Lace	0.19	0.50	RB	d	θ = 0.14, P_{so} = 10.5 psi, Failed on next cycle, Fragment loose from mesh.
.42	0.[illegible]	Lace	0.42	0.42	RB	d	θ = 1.2, P_{so} = 87 psi, Destroyed on next loading.
.42	C.[illegible]	Lace	0.42	0.42	RB	d	θ = 1.52, P_{so} = 91 psi, Destroyed on next loading.

Table 3.4 Dynamic Box Tests

Element	Restraint	L/t	Midspan ρ	Midspan ρ'	Support ρ	Support ρ'	f_y (ksi)	f_c' (ksi)	d (in)	t (in)	s/t	d_b (in)	Shea Reinfor
1/8-MC-71	4 sides	10.0	0.42	0.42	0.42	0.42	91.5	5.7	--	3.00	0.42	0.14	Lac
F1-77	2 sides	12	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F2-77	2 sides	12	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F3-77	2 sides	12	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F4-77	2 sides	12	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F5-77	2 sides	12	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F6-77	2 sides	12	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F7-77	2 sides	12	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F8-77	2 sides	9	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F9-77	2 sides	9	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F10-77	2 sides	9	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F11-77	2 sides	9	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F12-77	2 sides	9	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F13-77	2 sides	6	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F14-77	2 sides	6	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F15-77	2 sides	6	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F16-77	2 sides	18	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F17-77	2 sides	18	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F18-77	2 sides	18	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F19-77	2 sides	18	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F20-77	2 sides	18	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F21-77	4 sides	18	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F22-77	4 sides	18	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
F23-77	4 sides	18	2.0	2.0	2.0	2.0	60	6	--	4.00	1.00	0.50	Non
FH1-78	4 sides	8.6	1.0	1.0	1.0	1.0	75	7.0	4.80	5.60	0.71	0.50	doub
FH2-78	4 sides	8.6	1.0	1.0	1.0	1.0	57	7.6	4.80	5.60	0.71	0.50	135-
FH3-78	4 sides	8.6	1.0	1.0	1.0	1.0	57	7.8	4.80	5.60	0.71	0.50	135-
FH4-79	4 sides	8.6	1.0	1.0	1.0	1.0	65	6.7	4.80	5.60	0.71	0.50	135-
FH5-79	4 sides	8.6	1.5	1.5	1.5	1.5	69	6.1	12.00	13.50	0.41	0.50	135-
FH6-79	4 sides	8.6	1.0	1.0	1.0	1.0	65	6.8	4.80	5.60	0.71	0.50	135-
FH7-79	3-bay	8.6	1.0	1.0	1.0	1.0	71	5.1	4.80	5.60	0.71	0.50	135-
DS1-81	2 sides	8.6	1.0	1.0	1.0	1.0	63	3.9	4.80	5.60	0.71	0.50	doub
DS2-81	2 sides	8.6	1.0	1.0	1.0	1.0	63	3.9	4.80	5.60	0.71	0.50	135-
DS3-81	2 sides	8.6	1.0	1.0	1.0	1.0	63	4.0	4.80	5.60	0.71	0.50	135-
DS4-81	2 sides	8.6	1.0	1.0	1.0	1.0	63	5.9	4.80	5.60	0.71	0.50	135-
DS5-81	2 sides	8.6	1.0	1.0	1.0	1.0	63	6.0	4.80	5.60	0.71	0.50	135-
DS1-82	2 sides	6.2	0.75	0.75	0.75	0.75	80	7.0	6.40	7.30	0.55	0.50	135-
DS2-82	2 sides	6.2	0.75	0.75	0.75	0.75	80	7.7	6.40	7.30	0.55	0.50	135-
DS3-82	2 sides	6.2	0.75	0.75	0.75	0.75	80	7.5	6.40	7.30	0.55	0.50	135-
DS4-82	2 sides	6.2	1.20	1.20	1.20	1.20	67	7.4	6.40	7.30	0.55	0.63	135-
DS5-82	2 sides	6.2	1.20	1.20	1.20	1.20	67	7.8	6.40	7.30	0.55	0.63	135-
DS6-82	2 sides	6.2	1.20	1.20	1.20	1.20	67	7.3	6.40	7.30	0.55	0.63	135-
SB1-82	rigid	8.3	0.50	0.50	0.50	0.50	90.2	6.9	2.40	2.90	0.69	0.18	closed
SB2-82	rigid	8.3	0.50	0.50	0.50	0.50	90.2	6.9	2.40	2.90	0.69	0.18	closed
F1-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	6.2	1.94	2.50	1.50	0.25	double
F2-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.3	1.94	2.50	1.50	0.25	double
F3-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.0	1.94	2.50	1.50	0.25	double
F4-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.0	1.94	2.50	1.50	0.25	double
F5-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.1	1.94	2.50	1.50	0.25	double
F6-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	3.2	1.94	2.50	1.50	0.25	double
F7-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	4.1	1.94	2.50	1.50	0.25	double
F8-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.3	1.94	2.50	1.50	0.25	double

Legend for Miscellaneou

DOB = depth of burial
3-H = 3-hinged mechanism
3-HM = 3-hinged mechanism membrane
b = undetermined

P_{so} = peak surface
MD = Medium Damag
incipient f
c = collapse
r = pulled out

	f_c' ksi)	d (in)	t (in)	s/t	d_b (in)	Shear Reinforcement	ρ_s %	s_s/t	θ	Δ_{perm}	Remarks
it	5.7	--	3.00	0.42	0.14	Lace					Wall, z = 0.50, no steel failed, heavy damage
	6	--	4.00	1.00	0.50	None	--	--	0.47	0.2	Z = 2.40, open-end box, buried wall, sand, C- no damage
	6	--	4.00	1.00	0.50	None	--	--	1.40	0.6	Z = 1.80, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	7.10	3.0	Z = 1.20, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	2.40	1.0	Z = 2.00, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	c	c	Z = 1.50, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	0.70	0.3	Z = 2.30, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	15.20	8.5	Z = 1.90, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	1.00	0.3	Z = 1.80, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	10.40	3.3	Z = 1.20, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	0	0	Z = 2.00, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	29.10	10.0	Z = 1.50, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	1.60	0.5	Z = 1.90, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	29.10	10.0	Z = 1.40, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	0	0	Z = 1.50, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	7.10	1.5	Z = 1.00, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	26.60	6.0	Z = 0.75, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	c	c	Z = 0.50, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	1.80	1.1	Z = 2.80, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	10.20	6.5	Z = 2.30, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	c	c	Z = 2.30, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	0	0	Z = 1.86, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	2.20	1.4	Z = 1.16, open-end box, buried wall, sand, C-
	6	--	4.00	1.00	0.50	None	--	--	c	--	Z = 0.70, open-end box, buried wall, sand, C-
	7.0	4.80	5.60	0.71	0.50	double	1.50	0.71	1.20	0.50	DOB = L/2, 3-H, P_{so} = 1812, no steel broken
	7.6	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	c	c	DOB = L/2, S, all steel broken at supports,
1	7.8	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	14.00	6.00	DOB = L/2, 3-H, P_{so} = 2176, 5% tension at mid
1	6.7	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	26.60	12.00	DOB = L/5, 3-H, P_{so} = 1900, 10% tension at mi
1	6.1	12.00	13.50	0.41	0.50	135-S-90	1.50	0.41	7.50	3.25	DOB = L/5, S, P_{so} = 11,500, no steel broken
1	6.8	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	c	c	DOB = L/2, 3-HM, P_{so} = 8052, 60% ten at midsp
1	5.1	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	c	c	DOB = L/5, 3-HM, P_{so} = 2364, 95% tension and
1											
1	3.9	4.80	5.60	0.71	0.50	double	1.50	0.71	c	c	DOB = L/5, S, P_{so} = 4109, 27% ten & 14% comp
	3.9	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	c	c	DOB = L/5, S, P_{so} = 5664, 9% ten rupture at
1	4.0	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	22.60	10.00	DOB = L/5, S, P_{so} = 3333, no steel rupture
1	5.9	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	c	c	DOB = L/5, S, P_{so} = 4031, 73% ten & 45% comp
1	6.0	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	c	c	DOB = L/5, S, P_{so} = 6025, 68% tension & 55%
1											
1	7.0	6.40	7.30	0.55	0.50	135-S-90	0.50	0.22	c	c	DOB = L/5, S, P_{so} = 7624, 29% tension & 14%
	7.7	6.40	7.30	0.55	0.50	135-S-90	0.50	0.22	c	c	DOB = L/5, S, P_{so} = 5682, 46% tension & 21%
0	7.5	6.40	7.30	0.55	0.50	135-S-90	0.50	0.22	10.40	4.13	DOB = L/5, S, P_{so} = 3448, no steel broken
0	7.4	6.40	7.30	0.55	0.63	135-S-90	0.50	0.22	c	c	DOB = L/5, S, P_{so} = 8875, 7% tension rupture
0	7.8	6.40	7.30	0.55	0.63	135-S-90	0.50	0.22	28.20	12.00	DOB = L/5, S, P_{so} = 5034, no steel broken
0	7.3	6.40	7.30	0.55	0.63	135-S-90	0.50	0.22	8.90	3.50	DOB = L/5, S, P_{so} = 3377, no steel broken
0.											
0	5.9	2.40	2.90	0.69	0.18	closed hoop	0.25	0.69	b	b	DOB = L/2, P_{so} = 3300, steel rupture undeter
	5.9	2.40	2.90	0.69	0.18	closed hoop	0.25	0.69	3.60	0.75	DOB = L/2, P_{so} = 800, no steel broken
0.											
0.	5.2	1.94	2.50	1.50	0.25	double 135	0.18	0.60	0.90	0.25	DOB = 4L/11, 3-H, P_{so} = 127, no steel broken
	5.3	1.94	2.50	1.50	0.25	double 135	0.18	0.60	c	c	DOB = 0, P_{so} = 129, 100% tens & comp @ midsp failed near midheight
0.											
0.	5.0	1.94	2.50	1.50	0.25	double 135	0.18	0.60	0.20	0.06	DOB = 4L/11, P_{so} = 34, no steel broken
	5.0	1.94	2.50	1.50	0.25	double 135	0.18	0.60	1.70	0.50	DOB = 4L/11, P_{so} = 142, no steel broken
0.	5.1	1.94	2.50	1.50	0.25	double 135	0.18	0.60	3.10	0.88	DOB = 4L/11, P_{so} = 158, no steel broken
0.	3.2	1.94	2.50	1.50	0.25	double 135	0.18	0.60	2.40	0.69	DOB = 4L/11, P_{so} = 141, no steel broken
0.	4.1	1.94	2.50	1.50	0.25	double 135	0.18	0.60	2.30	0.66	DOB = 4L/11, P_{so} = 134, no steel broken
0.	5.3	1.94	2.50	1.50	0.25	double 135	0.18	0.60	1.70	0.50	DOB = 4L/11, P_{so} = 134, no steel broken
0.											
0.											

Legend for Miscellaneous Symbols

P_{so} = peak surface overpressure
MD = Medium Damage - less than
incipient failure, light spalling
membrane c = collapse
r = pulled out

Remarks

el failed, heavy damage

, buried wall, sand, C-4 cylindrical charge,

, buried wall, sand, C-4, small cracks
, buried wall, sand, C-4, major damage, near breach
, buried wall, sand, C-4, small cracks
, buried wall, sand, C-4, breach
, buried wall, sand, C-4, small cracks
, buried wall, sand, C-4, breach
, buried wall, sand, C-4, small cracks
, buried wall, sand, C-4, major damage, near breach
, buried wall, sand, C-4, small cracks
, buried wall, sand, C-4, breach
, buried wall, sand, C-4, small cracks
, buried wall, sand, C-4, no comment
, buried wall, sand, C-4, slight cracks
, buried wall, sand, C-4, cracked concrete
, buried wall, sand, C-4, severe concrete damage
, buried wall, sand, C-4, breach
, buried wall, sand, C-4, cracks
, buried wall, sand, C-4, breach
, buried wall, sand, C-4, breach
, buried wall, sand, C-4, slight cracks
, buried wall, sand, C-4, rear spalling
, buried wall, sand, C-4, breach

1812, no steel broken
el broken at supports, none at midspan, P_{so} = 9000
2176, 5% tension at midspan rupture
1900, 10% tension at midspan, 40% ten & 20% comp @ supp
,500, no steel broken
8052, 60% ten at midspan, 95% ten & 45% comp at supp
2364, 95% tension and compression rupture at support

09, 27% ten & 14% comp at support rupt, remain bars r
64, 9% ten rupture at supp remaining bars pulled out
33, no steel rupture
31, 73% ten & 45% comp rupt at support, remain bars r
25, 68% tension & 55% comp rupture at support

24, 29% tension & 14% comp rupture at support
82, 46% tension & 21% comp rupture at support
48, no steel broken
75, 7% tension rupture at support, remaing bars pull
34, no steel broken
77, no steel broken

steel rupture undetermined
no steel broken

= 127, no steel broken
0% tens & comp @ midspan MD, 100% tens @ support r wall
height
no steel broken
, no steel broken
, no steel broken
, no steel broken
, no steel broken
, no steel broken

Table 3.4 Dynamic Box Tests (continued)

Element	Restraint	L/t	Midspan ρ	Midspan ρ'	Support ρ	Support ρ'	f_y (ksi)	f_c' (ksi)	d (in)	t (in)	s/t	d_b (in)	Rei
F1-84	2 sides	14.7	1.20	0.40	1.60	1.14	63.5	3.1	1.59	2.25	1.00	0.20	
F2-84	2 sides	14.7	1.20	0.40	1.60	1.14	63.5	3.2	1.59	2.25	1.00	0.20	
F3-84	2 sides	14.7	1.20	0.40	1.60	1.14	63.5	3.0	1.59	2.25	1.00	0.20	
F4-84	2 sides	14.7	1.20	0.40	1.60	1.14	63.5	3.3	1.59	2.25	1.00	0.20	
B-84	2 sides	14.8	0.51	0.51	0.51	0.51		4.3	5.38	6.00	0.67	0.18	1
B4-85	2 sides	10	0.50	0.50	0.50	0.50	74.4	5.6	3.57	4.30	0.69	0.25	1
B5-85	2 sides	10	0.50	0.50	0.50	0.50	74.4	6.4	3.57	4.30	0.69	0.25	1
B5A-85	2 sides	10	0.50	0.50	0.50	0.50	74.4	6.1	3.57	4.30	0.69	0.25	1
B6-85	2 sides	10	1.00	1.00	1.00	1.00	63.4	6.4	3.41	4.30	0.69	0.38	1
B6A-85	2 sides	10	1.00	1.00	1.00	1.00	63.4	6.4	3.41	4.30	0.69	0.38	1
B7-85	2 sides	5	0.50	0.50	0.50	0.50	63.4	5.7	7.74	8.60	0.35	0.38	1
B7A-85	2 sides	5	0.50	0.50	0.50	0.50	63.4	5.7	7.74	8.60	0.35	0.38	1
B8-85	2 sides	10	0.50	0.50	0.50	0.50	74.4	5.6	3.57	4.30	0.69	0.25	1
B8A-85	2 sides	10	0.50	0.50	0.50	0.50	74.4	5.6	3.57	4.30	0.69	0.25	1
B9-85	2 sides	10	0.50	0.50	0.50	0.50	74.4	6.0	3.57	4.30	0.69	0.25	1
B10-85	2 sides	10	0.50	0.50	0.50	0.50	74.4	6.0	3.57	4.30	0.69	0.25	1
KW-87	4 sides one-way	12.9	1.10	0.36	1.10	0.36	61.6	3.0	7.40	10.30	0.58	0.75	
H1-89	2 sides	10	1.0	1.0	1.0	1.0	67.4	6.1	3.41	4.30	0.70	0.38	1
H2-89	2 sides	5	0.5	0.5	0.5	0.5	67.4	6.4	7.74	8.60	0.35	0.38	1
H3-89	2 sides	10	1.0	1.0	1.0	1.0	67.4	5.9	3.41	4.30	0.70	0.38	1
H4-89	2 sides	10	1.0	1.0	1.0	1.0	67.4	5.9	3.41	4.30	0.70	0.38	1

Legend for Miscellaneous S

DOB = depth of burial
3-H = 3-hinged mechanism
3-HM = 3-hinged mechanism membrane
b = undetermined

P_{so} = peak surface ove
MD = Medium Damage -
incipient failu
c = collapse

ar rcement	ρ	t (in)	s/t	d_b (in)	Shear Reinforcement	ρ_s %	s_s/t	θ	Δ_{perm}	Remarks
-	-	2.25	1.00	0.20	--	--	--	1.40	0.41	DOB = 4L/11, P_{so} = 120, steel rupture at support
-	-	2.25	1.00	0.20	--	--	--	15.30	4.50	DOB = 4L/11, P_{so} = 184, 100% tension at midspan r
-	-	2.25	1.00	0.20	--	--	--	29.90	9.50	DOB = 4L/11, P_{so} = 128, 100% tension at midspan r
-	-	2.25	1.00	0.20	--	--	--	0.90	0.25	DOB = 4L/11, P_{so} = 162, steel rupture undetermine
S-90	0.	6.00	0.67	0.18	135-S-90	0.31	0.67	16.00	11.40	Third layer of steel at mid-depth; Ps=0.16, near or straightend at support. Tension Memr. No pr
S-135	0.	4.30	0.69	0.25	135-S-135	0.32	0.59	1.01	0.13	Z = 4.0, max defl = 0.38, buried wall, external
S-135	0.	4.30	0.69	0.25	135-S-135	0.32	0.59	5.66	1.50	Z = 2.0, max defl = 2.13, buried wall, external
S-135	0.	4.30	0.69	0.25	135-S-135	0.32	0.59	c	c	Z = 1.5, max defl = breach, buried wall, externa
S-135	0.	4.30	0.69	0.38	135-S-135	0.50	0.59	4.15	1.00	Z = 2.0, max defl = 1.56, buried wall, external
S-135	0.	4.30	0.69	0.38	135-S-135	0.50	0.59	9.58	2.75	Z = 1.5, max defl = 3.63, buried wall, external
S-135	0.	8.60	0.35	0.38	135-S-135	0.27	0.35	0.83	0.38	Z = 2.0, max defl = 0.63, buried wall, external
S-135	0.	8.60	0.35	0.38	135-S-135	0.27	0.35	4.65	2.88	Z = 1.0, max defl = 3.50, buried wall, external
S-135	0.	4.30	0.69	0.25	135-S-135	0.32	0.59	2.66	0.63	Z = 2.0, max defl = 1.00, buried wall, external
S-135	0.	4.30	0.69	0.25	135-S-135	0.32	0.59	2.50	0.44	Z = 2.0, max defl = 0.94, buried wall, external
S-135	0.	4.30	0.69	0.25	135-S-135	0.32	0.59	6.84	2.13	Z = 2.0, max defl = 2.58, buried wall, external
S-135	0.	4.30	0.69	0.25	135-S-135	0.32	0.59	4.49	1.19	Z = 2.0, max defl = 1.69, buried wall, external
ne	-	10.30	0.58	0.75	None	--	--	14.00	17.00	Full scale, thin steel decking on bottom surface
S-135	0.	4.30	0.70	0.38	135-S-135	0.50	0.70	c	c	Z = 2.0, wall buried in reconstituted clay, brea broken bars
S-135	0.	8.60	0.35	0.38	135-S-135	0.28	0.26	2.10	1.19	Z = 2.0, wall buried in reconstituted clay, ligh max defl = 1.56"
S-135	0.	4.30	0.70	0.38	135-S-135	0.50	0.70	3.80	1.13	Z = 2.0, wall buried in compacted sand, light da max defl = 1.44"
S-135	0.	4.30	0.70	0.38	135-S-135	0.50	0.70	26.40	9.19	Z = 2.0, wall buried in in-situ clay, most tens max defl = 10.69, tens. membrane, most

Legend for Miscellaneous Symbols

sure
than
light spalling ism

ism membrane

P_{so} = peak surface overpressure
MD = Medium Damage - less than
incipient failure, *light spalling*
c = collapse

Remarks

termined
re, undete 20, steel rupture at support undetermined
re, undete 84, 100% tension at midspan rupture, undetermined support
support 28, 100% tension at midspan rupture, undetermined support
.62, steel rupture undetermined at support

span. Sti 1 at mid-depth; Ps=0.16, near midspan. Stirrups rupture
pal steel upport. Tension Memr. No principal steel rupture.
, response 0.38, buried wall, external shot, response mode undefined
, hinged m 2.13, buried wall, external shot, hinged mode
ot, flexur breach, buried wall, external shot, flexural mode
, undefine 1.56, buried wall, external shot, undefined mode
, flexure- 3.63, buried wall, external shot, flexure-membrane mode
, undefine 0.63, buried wall, external shot, undefined mode
, flexure- 3.50, buried wall, external shot, flexure-membrane mode
, flexure 1.00, buried wall, external shot, flexure mode
, flexure 0.94, buried wall, external shot, flexure mode
, flexure 2.58, buried wall, external shot, flexure mode
, flexure 1.69, buried wall, external shot, flexure mode

EST-160 psi eel decking on bottom surface; HEST-160 psi, DOB = 4"

(hole) with d in reconstituted clay, breach (hole) with 19" defl, many
s
amage, crac d in reconstituted clay, light damage, cracking,
1.56"
e, small cr d in compacted sand, light damage, small cracks,
1.44"
steel broke d in in-situ clay, most tension steel broken,
ar hung on 10.69, tens. membrane, most cover hung on

Table 3.5 Laterally-Restrained Boxes (Point-Source Loading)

Legend

U = not reported (unknown)
N = No
Y = Yes

z	L/t	θ	Shear Rein.	$s \leq t$	$s_s \leq t/2$	Damage
1.5	8	29	None	Y	---	Local Breach
1.4	8	28	None	Y	---	U
0.75	6	26	None	Y	---	U
1.9	12	15	None	Y	---	Local Breach
1.2	9	10	None	Y	---	Major Damage
1.5	10	10	135-s-135	Y	N	U
1.2	12	7	None	Y	---	Major Damage
1.0	6	7	None	Y	---	Slight
1.16	18	2	None	Y	---	Slight
1.8	12	2	None	Y	---	Slight
1.8	9	1	None	Y	---	Slight
1.86	18	0	None	Y	---	Slight
1.5	6	0	None	Y	---	Slight
1.0	5	5	135-s-135	Y	Y	U
1.9	9	2	None	Y	---	Slight
2.0	10	26	135-s-135	Y	N	U
2.3	18	10	None	Y	---	Local Breach
2.0	10	7	135-s-135	Y	N	U
2.0	10	6	135-s-135	Y	N	U
2.0	10	4.5	135-s-135	Y	N	U
2.0	10	4	135-s-135	Y	N	U
2.0	10	3.5	135-s-135	Y	N	Slight
2.0	10	2.5	135-s-135	Y	N	U
2.0	12	2.5	None	Y	---	Slight
2.0	10	2	135-s-135	Y	N	U
2.0	5	2	135-s-135	Y	Y	Slight
2.8	18	1.5	None	Y	---	Slight
4.0	10	1	135-s-135	Y	---	Slight
2.3	12	1	None	Y	---	Slight
2.0	5	1	135-s-135	Y	Y	U
2.4	12	0.5	None	Y	---	Slight
5.0	7	0.2	None	Y	---	Slight
2.0	9	0	None	Y	---	Slight

Table 3.6 Laterally-Restrained Boxes (HEST Loading)

Legend

U = not reported (unknown)
N = No
Y = Yes

L/t	θ	Shear Rein.	$s \le t$	$\rho \backslash \rho_s$	$s_s \le t/2$	Damage
6	11	135-s-90	Y	0.75\0.5	Y	steel not ruptured
6	9	135-s-90	Y	1.2\0.5	Y	steel not ruptured
8	8	135-s-90	Y	1.5\1.5	Y	steel not ruptured
8	4	closed-hoop	Y	0.5\0.25	---	steel not ruptured
13	3.1	double-leg	N	0.69\0.18	N	steel not ruptured
13	2.5	double-leg	N	0.69\0.18	N	steel not ruptured
13	2	double-leg	N	0.69\0.18	N	steel not ruptured
13	2	double-leg	N	0.69\0.18	N	steel not ruptured
8.5	1.5	double-leg	Y	1.0\1.5	N	steel not ruptured
15	1.5	None	N	1.2	---	< 10% steel ruptured
13	1	double-leg	N	0.69\0.18	N	steel not ruptured
15	1	None	N	1.2	---	< 10% steel ruptured
13	0.5	double-leg	N	0.69\0.18	N	steel not ruptured
15	30	None	N	1.2	---	near incipient collapse
6	28	135-s-90	Y	1.2\0.5	Y	steel not ruptured
8	26	135-s-90	Y	1.0\1.5	N	< 50% steel ruptured
8	22	135-s-90	Y	1.0\1.5	N	steel not ruptured
14	16	135-s-90	Y	0.51\0.31	N	steel not ruptured
15	15	None	N	1.2	---	> 50% steel ruptured
13	14	None	Y	1.1	---	steel not ruptured
8	14	135-s-90	Y	1.0\1.5	N	< 10% steel ruptured

Table 3.7 Nonlaced Slabs (Point-Source Loading)

Legend

U = not reported (unknown)
N = No
Y = Yes

z	L/t	Shear Rein.	$s \le t$	ρ	Laterally Restrained	Damage
1.7	8	None	Y	0.15	N	SD
1.7	8	None	Y	0.15	N	SD
1.65	8	None	Y	0.15	N	PD
1.6	6	None	N	0.65	U	PD
1.5	8	None	Y	0.15	U	TD
1.5	14	None	Y	0.40	U	SD
1.5	14	None	Y	0.40	U	HD
1.25	6	None	N	0.65	U	TD
1.25	6	None	N	0.44	U	HD
1.25	6	None	N	0.65	U	HD
1.25	6	None	N	0.65	U	PD
1.25	6	Looped	N	0.65	Y	HD
1.1	8	None	Y	0.15	N	MD
1.05	8	None	Y	0.15	N	PD
1.02	7	None	Y	0.15	U	TD
1.0	8	None	Y	0.15	N	TD
1.0	8	None	Y	0.15	N	TD
1.0	7	None	Y	0.15	Y	SD
1.0	6	None	N	0.65	U	TD
1.0	6	Looped	N	0.65	N	PD
0.8	6	None	N	0.65	U	TD
0.5	14	None	Y	0.40	U	TD
0.5	8	None	Y	0.15	N	TD
0.5	6	None	N	0.65	U	HD
0.5	6	None	N	0.44	U	TD
0.5	6	None	N	0.65	U	HD
0.5	6	None	N	0.65	U	TD
0.5	6	None	N	0.65	N	TD
0.5	4	None	N	2.70	N	MD
0.5	2	None	N	0.15	U	TD

Table 3.7 Nonlaced Slabs (Point-Source Loading) (continued)

Legend

U = not reported (unknown)
N = No
Y = Yes

z	L/t	Shear Rein.	s ≤ t	ρ	Laterally Restrained	Damage
1.1	20	None	Y	0.31	Y	θ = 10.4°; no steel failed, shear crack @ one support (MD)
0.69	20	180-s-180	Y	1.0	Y	θ = 12.2°; no steel failed (MD)
0.69	20	180-s-180	Y	[illegible].0	Y	θ = 10.1°; no steel failed (MD)
0.65	20	180-s-180	Y	1.5	Y	θ = 10.5°; no steel failed (MD)
0.65	20	180-s-180	Y	2.5	Y	θ = 4.8°; no steel failed (SD-MD)
2.0	8	None	Y	0.15	U	TD
2.6	8	None	Y	0.15	N	SD
2.6	8	None	Y	0.15	N	PD
2.62	8	None	Y	0.15	N	SD
3.5	14	None	Y	0.40	U	SD

Table 3.8 Laced Slabs (Point-Source Loading)

Legend

U = not reported (unknown)
N = No
Y = Yes

z	L/t	ρ	ρ_s	Restrained	Damage
1.5	6	0.65	0.15	Y	HD
1.25	6	0.65	0.40	U	MD
1.0	6	0.65	0.15	N	HD
1.0	6	0.65	0.40	N	HD
1.0	6	0.65	0.15	Y	HD
1.0	6	0.65	0.15	Y	PD
1.0	6	2.70	1.20	Y	HD
0.9	6	2.70	1.20	Y	HD
0.8	6	0.65	0.15	N	PD
0.8	6	0.65	0.40	N	MD
0.5	6	0.65	0.40	U	HD
0.5	6	0.65	0.15	U	PD
0.5	6	0.65	0.40	U	PD
0.5	6	2.70	1.20	N	HD
0.5	4	2.70	1.20	Y	HD
0.5	2	0.69	0.53	U	MD
0.4	6	0.65	0.40	U	HD
0.4	1.8	0.65	0.53	U	HD
0.35	2	2.70	1.20	Y	HD
0.3	2	2.70	1.20	Y	HD
0.3	2	2.70	1.20	Y	PD

Table 3.9 Nonlaced Slabs (Static Loading)

Legend

U = not reported (unknown)
N = No
Y = Yes

θ	L/t	Shear Rein.	$s \le t$	$s_s \le t/2$	ρ/ρ_s	Damage
11.2	15	135-s-90	N	N	1.14/0.18	> 50% tension steel ruptured
12.6	10	135-s-90	N	N	0.74/0.18	< 50% tension steel ruptured
13	10	135-s-135	N	N	0.74/0.09	> 50% tension steel ruptured
14	10	double-leg	N	N	0.74/0.19	> 50% tension steel ruptured
14	10	135-s-135	N	N	0.74/0.18	> 50% tension steel ruptured
14	10	135-s-90	N	N	0.74/0.18	> 50% tension steel ruptured
14	10	None	N	N	1.58	No steel ruptured
14.5	10	135-s-135	N	N	0.74/0.18	> 50% tension steel ruptured
14.5	15	135-s-90	N	N	1.47/0.24	No steel ruptured
15	15	135-s-90	N	N	1.47/0.24	No steel ruptured
15.5	10	135-s-135	N	N	0.74/0.18	> 50% tension steel ruptured
16	15	135-s-90	N	N	0.58/0.18	> 50% tension steel ruptured
16.5	10	135-s-90	N	N	1.06/0.27	< 50% tension steel ruptured
16.5	10	None	N	N	.74	> 50% tension steel ruptured
16.5	10	135-s-135	Y	N	0.75/0.19	> 50% tension steel ruptured
16.7	15	135-s-90	N	N	1.14/0.18	No steel ruptured
17	10	135-s-90	N	N	0.52/0.22	> 50% tension steel ruptured
17	15	135-s-90	N	N	0.58/0.18	> 50% tension steel ruptured
18	10	135-s-90	N	N	0.74/0.18	< 50% tension steel ruptured
18	15	135-s-90	N	N	1.14/0.18	No steel ruptured
18	10	135-s-135	Y	Y	0.75/0.38	> 50% tension steel ruptured
18	10	None	N	N	0.74	> 50% tension steel ruptured
18.8	10	135-s-90	N	N	0.74/0.18	> 50% tension steel ruptured
19.5	10	135-s-90	N	Y	1.13/0.22	> 50% tension steel ruptured
19.5	10	135-s-90	N	N	0.52/0.22	> 50% tension steel ruptured
19.7	10	None	N	N	0.79	> 50% tension steel ruptured
19.7	10	None	N	N	1.13	< 50% tension steel ruptured
20	10	135-s-90	N	N	0.74/0.18	> 50% tension steel ruptured
20.5	10	135-s-135	N	Y	0.74/0.36	> 50% tension steel ruptured
20.5	10	None	N	N	1.14	< 50% tension steel ruptured
21	10	None	N	N	1.14	> 50% tension steel ruptured
22.5	10	None	N	N	1.13	< 50% tension steel ruptured

Table 3.9 Nonlaced Slabs (Static Loading) (continued)

Legend

U = not reported (unknown)
N = No
Y = Yes

θ	L/t	Shear Rein.	$s \le t$	$s_s \le t/2$	ρ/ρ_s	Damage
22.5	8.4	135-s-90	Y	N	1.02/1.53	> 50% tension steel ruptured
23.5	10	135-s-90	N	Y	1.13/0.22	> 50% tension steel ruptured
23.5	10	None	N	N	1.14	> 50% tension steel ruptured
23.5	10	135-s-135	N	N	1.13/0.06	> 50% tension steel ruptured
24	10	None	N	N	0.79	> 50% tension steel ruptured
24.5	10	135-s-90	N	Y	1.13/0.22	> 50% tension steel ruptured

Table 3.10 Laced Slabs (Static Loading)

Legend

U = not reported (unknown)
N = No
Y = Yes

θ	L/t	ρ/ρ_s	$s \le t$	$s_s \le t/2$	Laterally Restrained	Damage
.15	24	0.82/0.19	N	Y	Y	steel condition not reported
.16	24	2.11/1.37	Y	Y	Y	no steel ruptured
.19	24	0.89/0.42	N	Y	Y	steel condition not reported
.22	24	0.82/0.19	N	Y	Y	steel condition not reported
.23	24	0.82/0.19	N	Y	Y	steel condition not reported

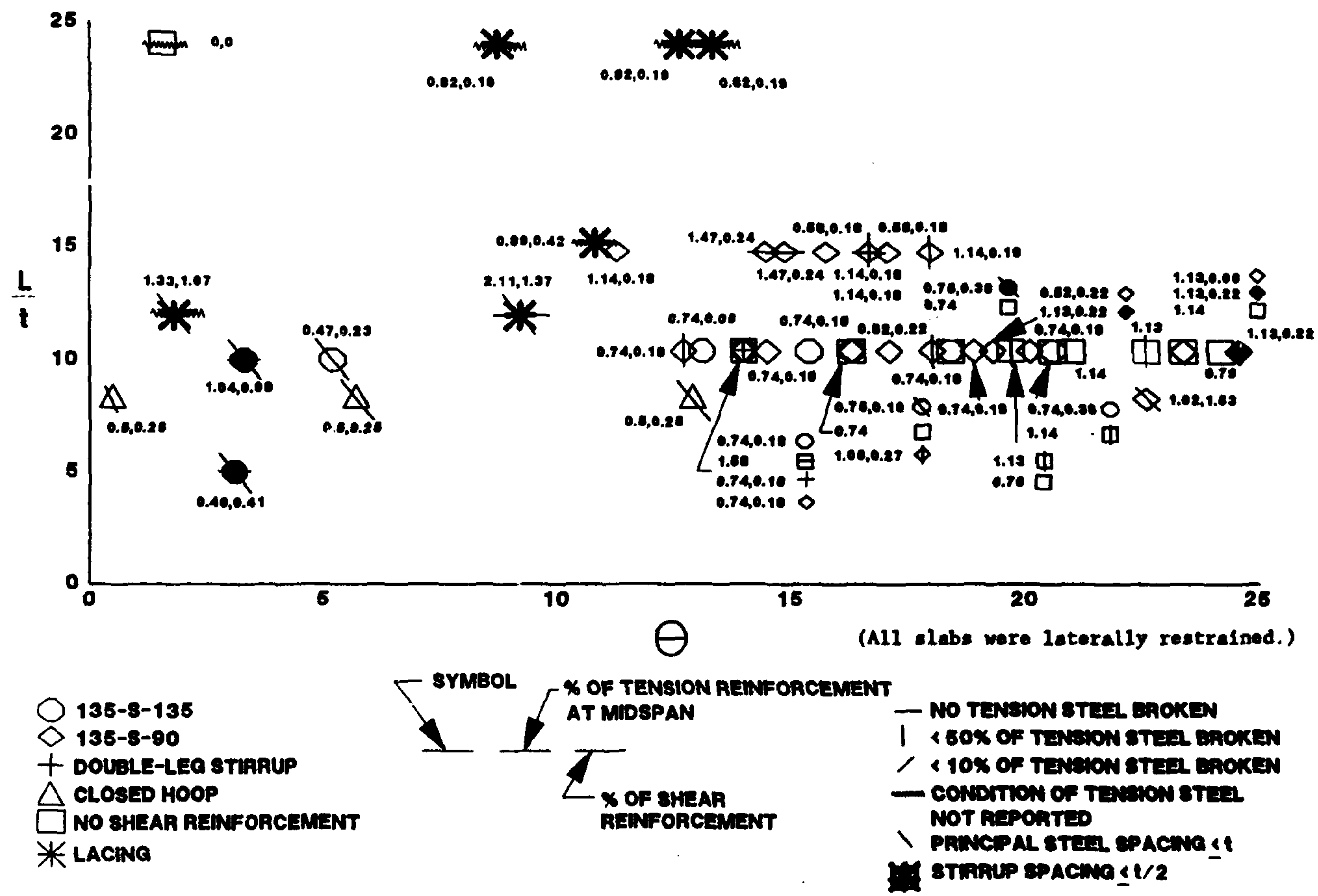


Figure 3.1. Statically-Loaded Slabs

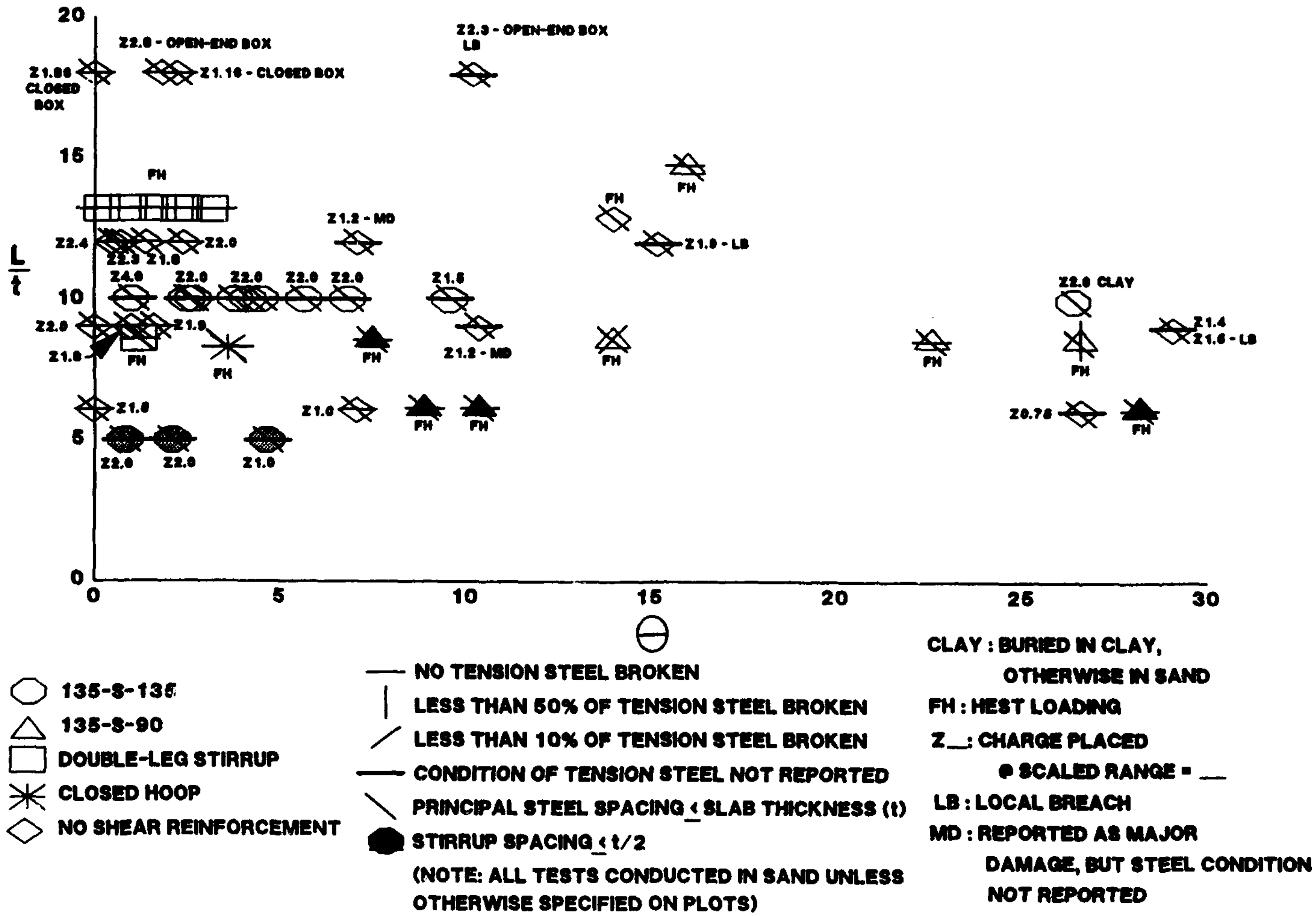


Figure 3.2. Dynamically-Loaded Slabs

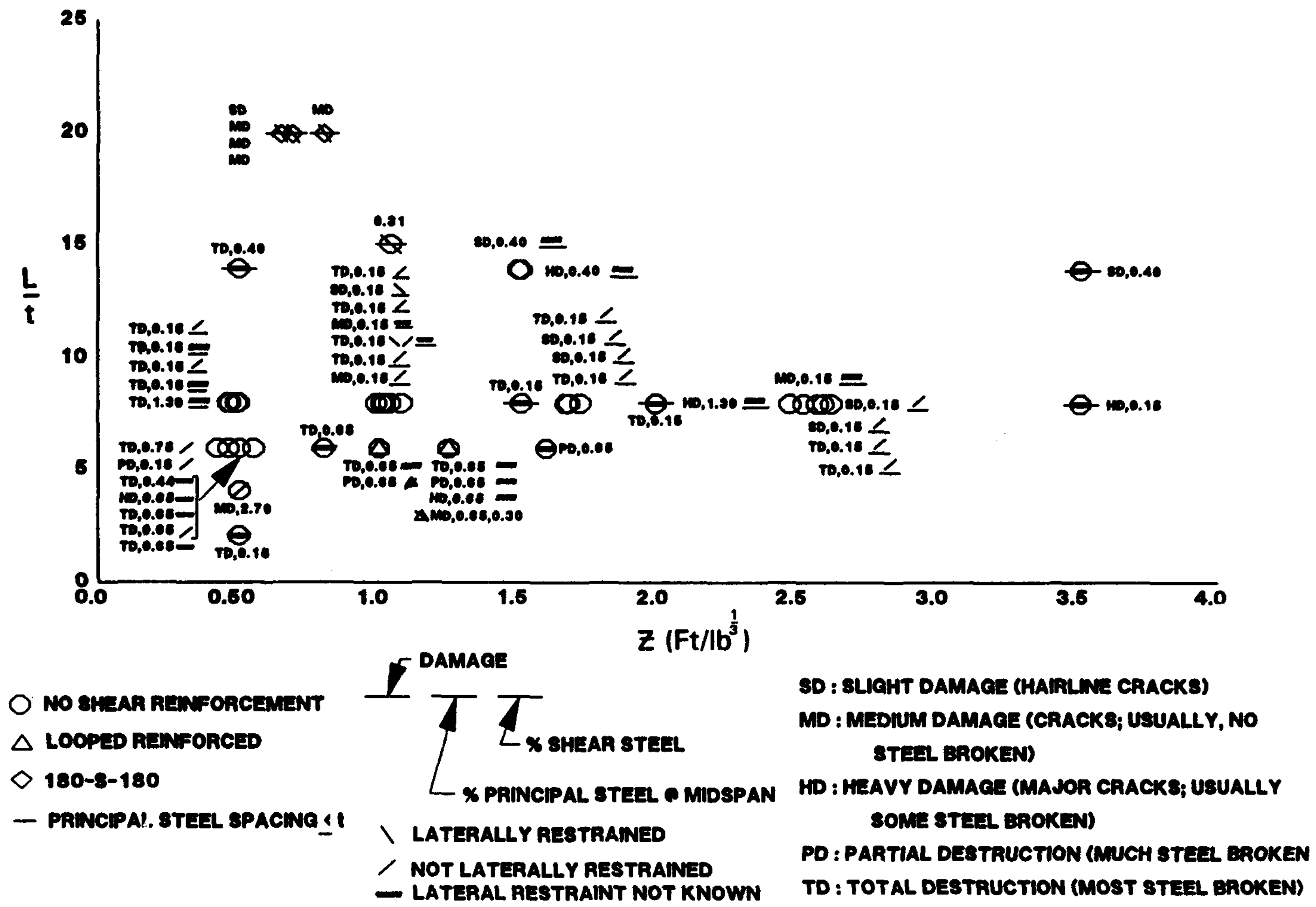


Figure 3.3. Dynamically-Tested Non-Laced Slabs

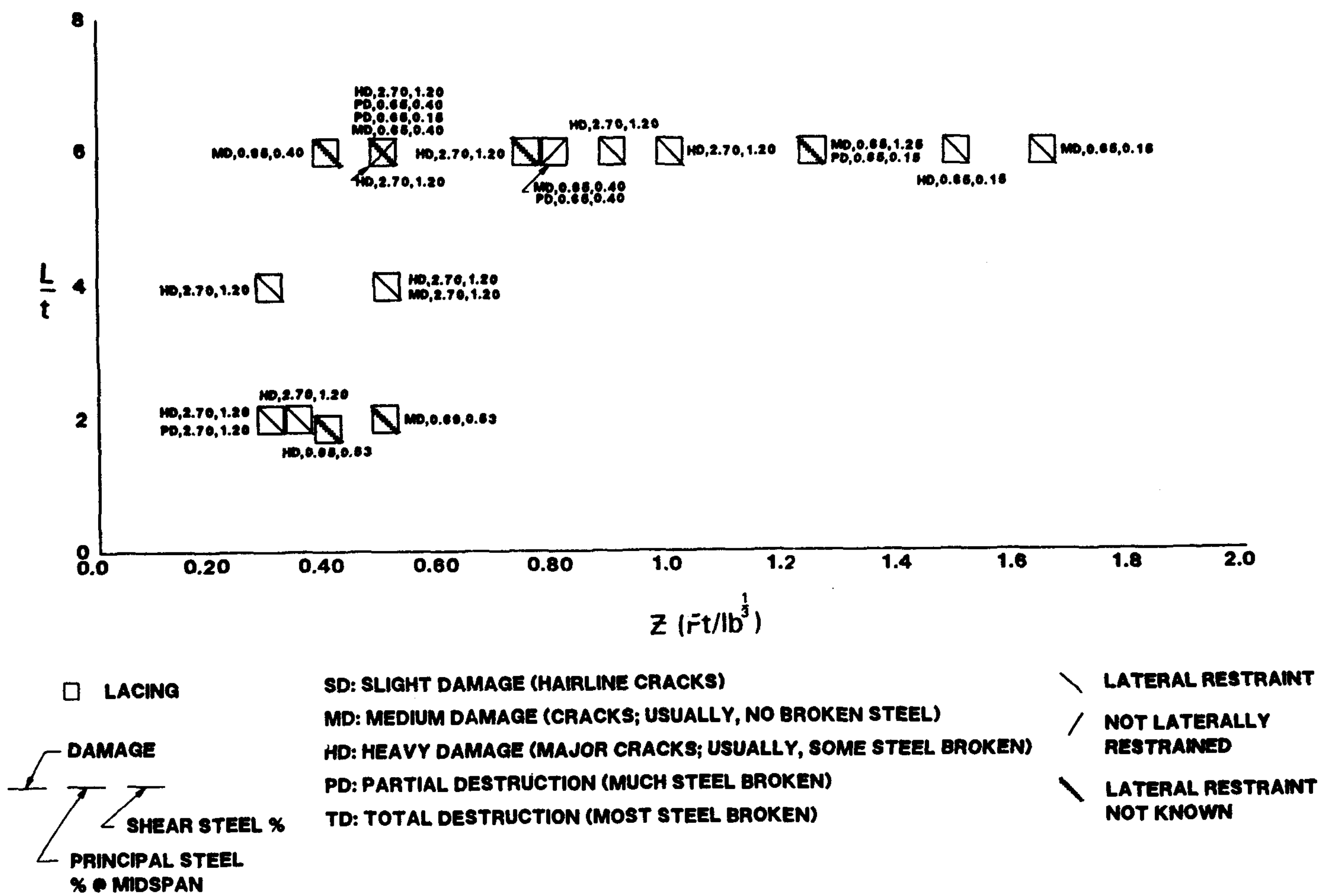


Figure 3.4. Dynamically-Tested Laced Slabs

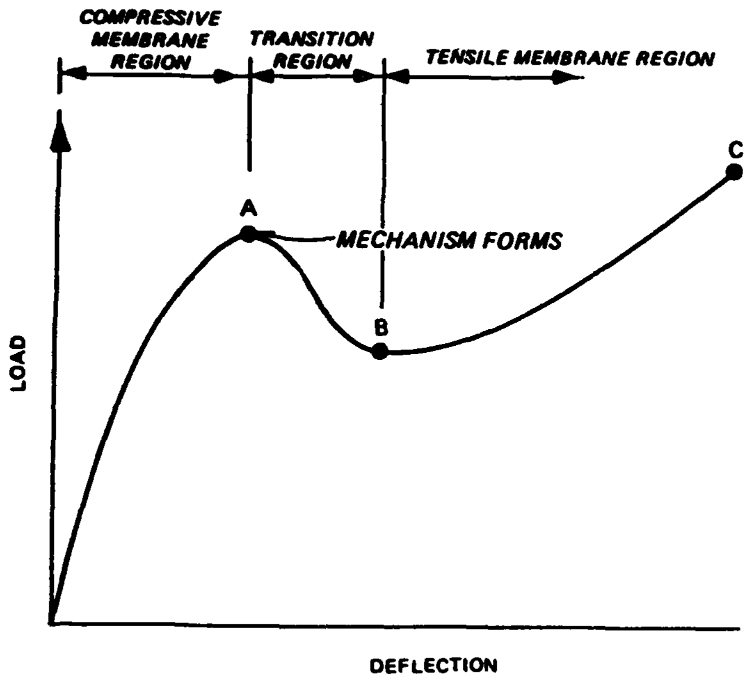


Figure 3.5. Load-Deflection Relationship for Restrained Slabs

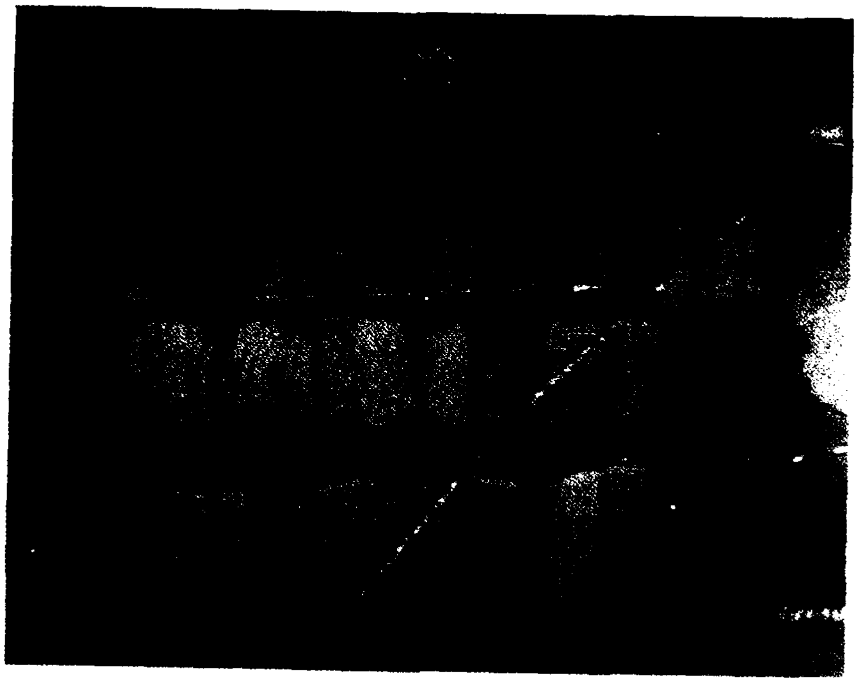


Figure 3.6. Posttest View of Slabs With Stirrups, W-83 Series

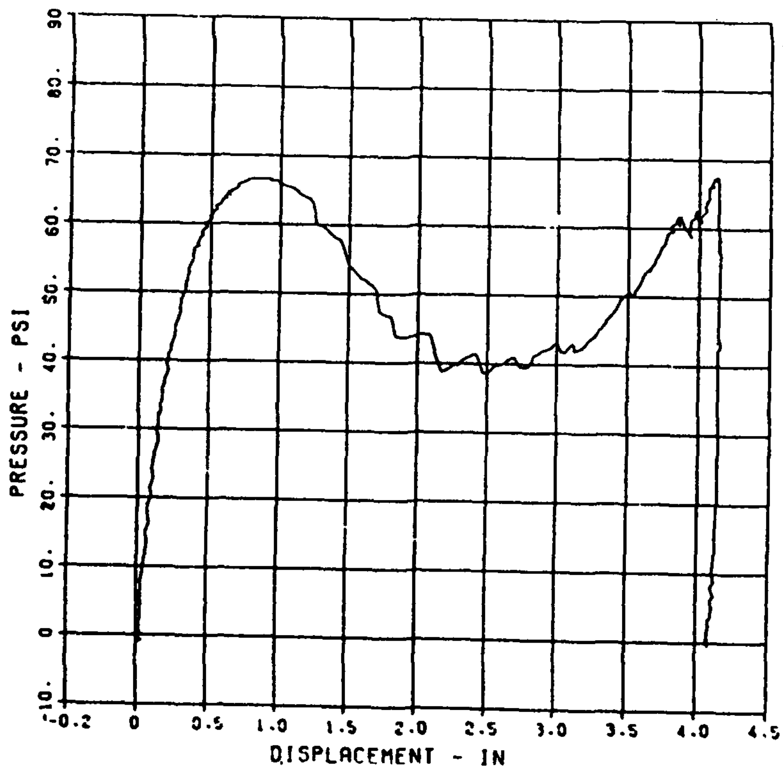


Figure 3.7. Load Deflection Curve for Close Stirrup Spacing

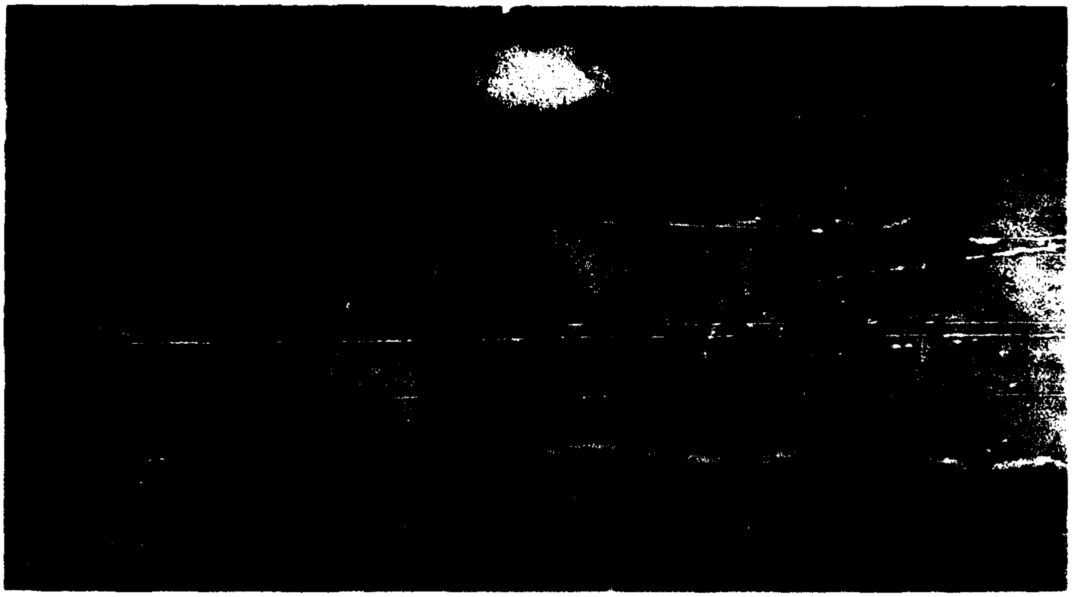


Figure 3.8. Posttest View of W-84 Series

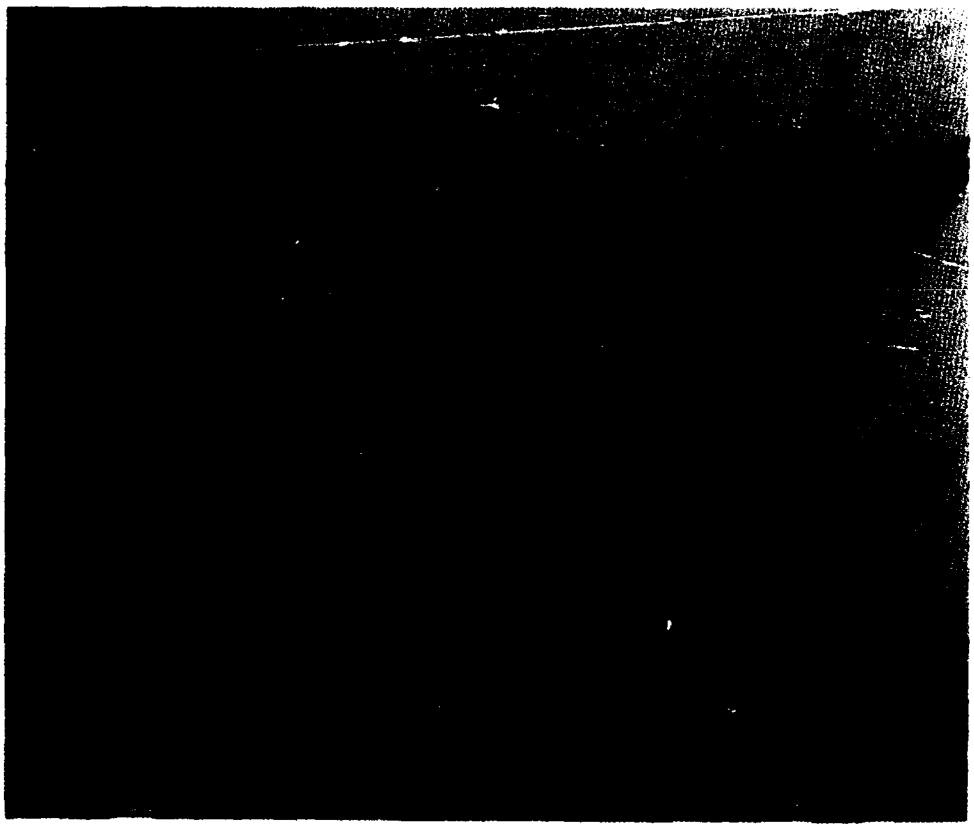


Figure 3.9. Damage to Structure Tested to Clay Backfill

Figure 3.10. Damage to Structure Tested to Sand Backfill

Figure 3.11. Interior View of Structure Tested in Sand Backfill

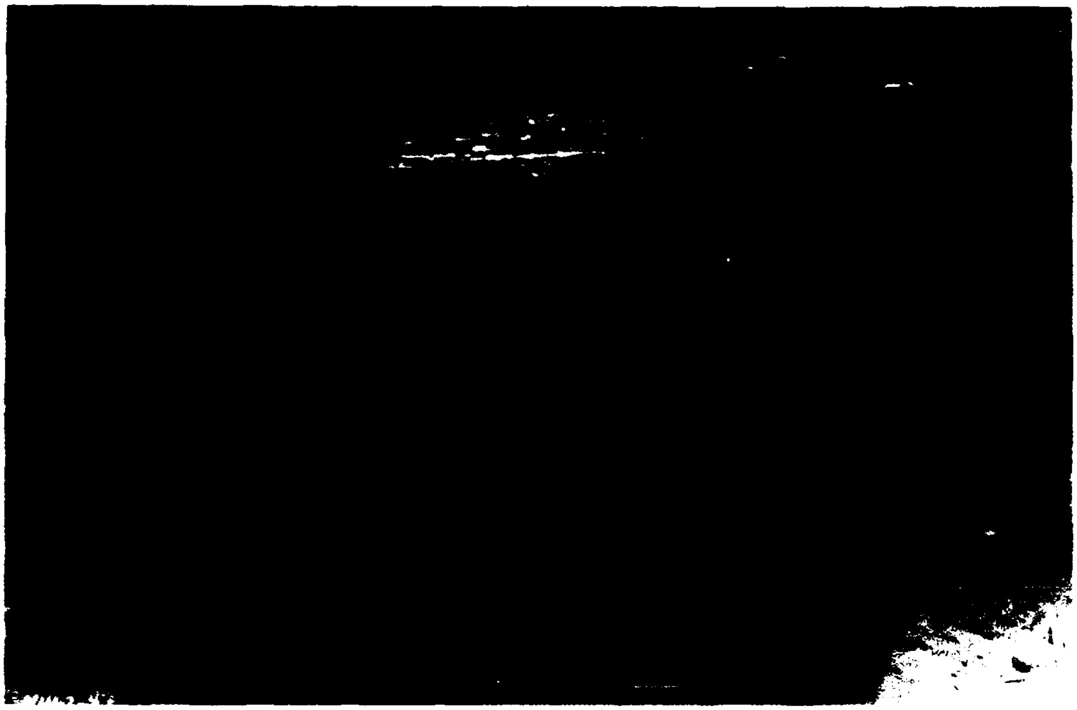


Figure 3.12. Shallow-buried Box With 10-inch Roof Deflections

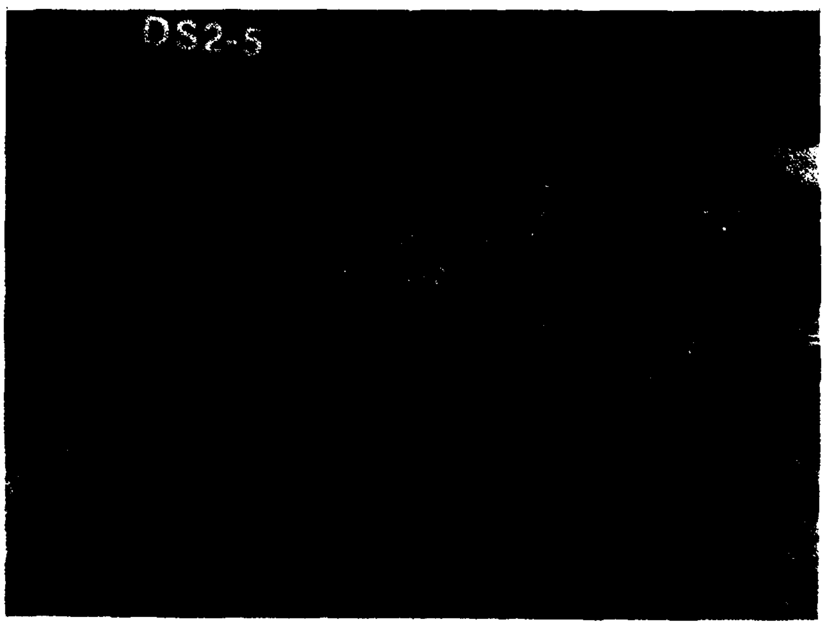


Figure 3.13. Shallow-Buried Box With 12-inch Roof Deflection

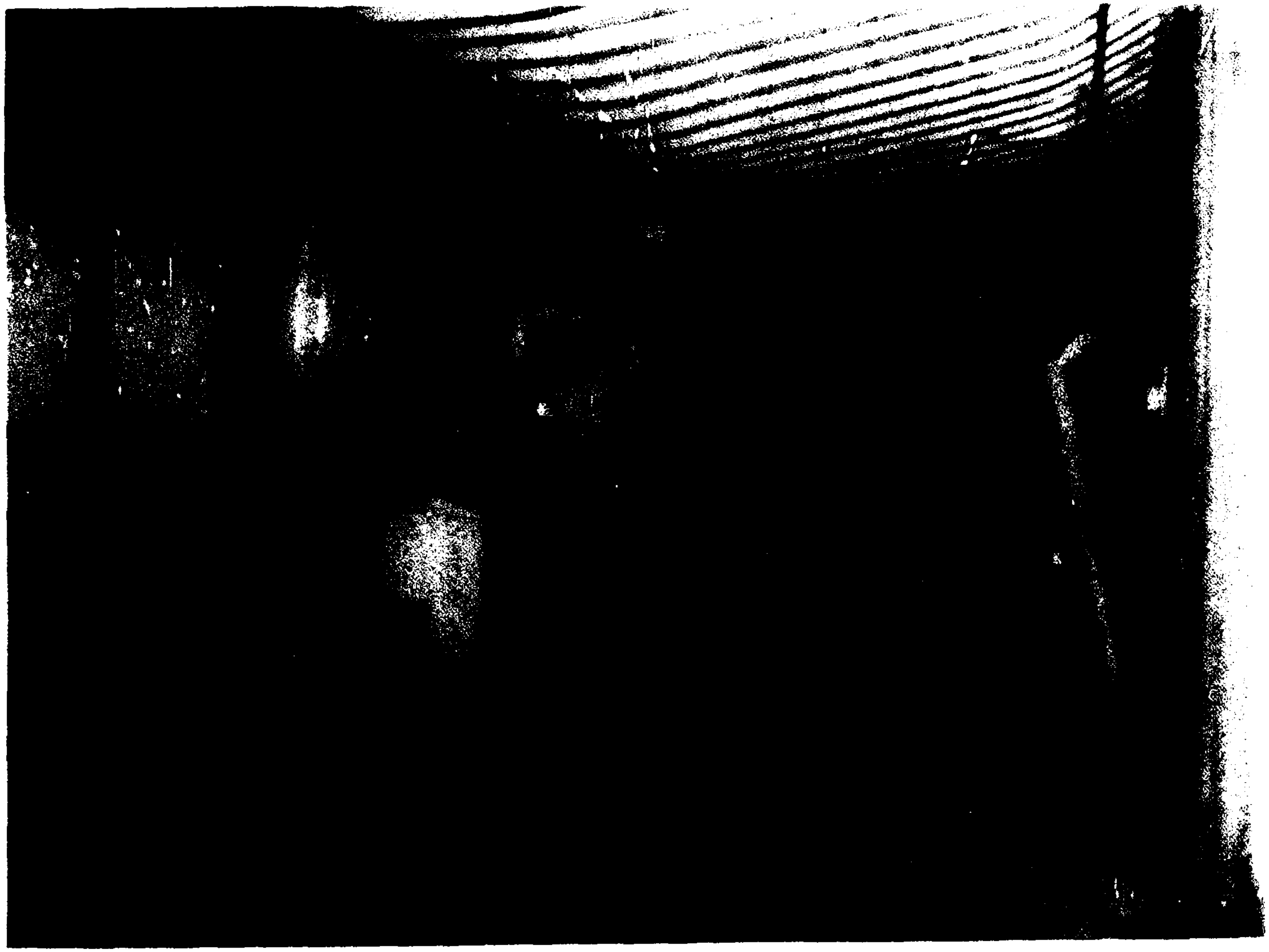


Figure 3.14. Interior View of KW-87

CHAPTER 4

EXPERIMENTAL DESCRIPTION

4.1 Introduction

Sixteen one-way reinforced concrete slabs were statically loaded at WES in May and June, 1991. The slabs were uniformly loaded with slowly changing water pressure to compare the behavior of laced and nonlaced slabs in a controlled laboratory environment. The review of current design criteria and data from previous studies as presented in preceding chapters indicates that, in addition to shear reinforcement details, the primary parameters that affect the large-deflection behavior of a one-way reinforced concrete slab include: support conditions, amount and spacing of principal reinforcement, scaled range (when subjected to blast loads), and the span-to-effective-depth (L/d) ratio. The effects of these parameters on the structural response of a slab must be considered in the study of the role of shear reinforcement. The following sections describe the slabs' construction details, the material properties, reaction structure, instrumentation, and the experimental procedure. The experimental results, along with discussion and analyses, are presented in subsequent chapters.

4.2 Construction Details

The slabs were designed to reflect the interaction of shear reinforcement details with the other primary parameters. The characteristics of each slab are qualitatively presented in Table 4.1. The same characteristics are presented in Table 4.2 in a quantitative manner, reflecting the practical designs based on

available construction materials. All slabs were designed to be supported in a clamped (longitudinally and rotationally restrained) condition. Each slab had a clear span of 24 inches, a width of 24 inches, and an effective depth of 2.4 inches, maintaining the L/d ratio at a value of 10. The slabs were 3 inches thick. The experimental program was designed to compare the effects of lacing bars and stirrups on slab behavior for three values of principal reinforcement ratio and three values of shear reinforcement spacing.

It was important that the ratio of principal steel spacing to slab effective depth (s/d) was held nearly constant among the slabs. Data from previous studies indicated that this ratio should be less than 1.0 in order to enhance the large-deflection behavior. The s/d ratio was maintained at a value of approximately 0.6. The shear reinforcement spacing was varied from a value equal to the effective depth (d) to approximately 3d/4 and d/2 (d/2 is the value typically given in design manuals for blast-resistant structures). It was impossible to maintain all of these design parameters at exact values using the reinforcement bar sizes available, but the variations were slight. For example, the purposely varied parameter between slabs no. 6 and 7 was the principal reinforcement ratio, while the shear reinforcement ratio category was "medium" for both slab no. 6 and slab no. 7. However, the actual shear reinforcement ratio values were 0.0034 and 0.0036 for slabs no. 6 and 7, respectively. The values of shear reinforcement ratio were identical when compared between a laced slab and a slab with stirrups for any category of principal reinforcement quantity. Figures 4.1 through 4.3 are

plan views showing slab proportions and the principal steel and temperature steel layouts for each of the slabs.

The temperature (transverse) steel spacing was identical for all of the slabs, but one difference in the temperature steel placement occurred between laced and nonlaced slabs. The temperature steel is typically placed exterior to the principal steel in laced slabs, but it is placed interior to the principal steel in the slabs having stirrups or no shear reinforcement. One exception was slab no. 13 (contained stirrups) in which the temperature steel was placed exterior to principal steel, thereby providing a correlation of the effect of this parameter being different for laced and nonlaced slabs.

Figures 4.4, 4.5, and 4.6 are sectional views cut through the lengths of the laced slabs. The dashed lacing bar in each figure indicates the configuration of the lacing bar associated with the next principal steel bar. The positions of the lacing bars were alternated to encompass all temperature steel bars. However, some temperature steel bars were not encompassed by lacing bars in slabs no. 4 and 5 due to the spacing of the lacing bar bends. The spacings of the lacing bar bends were controlled by the shear reinforcement quantities in corresponding slabs with stirrups. Figures 4.7 through 4.10 are sectional views cut through the lengths of the slabs with stirrups. Figure 4.11 shows typical stirrup details for slabs with D3 principal reinforcement. The stirrups for slabs with D1 principal steel were similar, differing slightly in length due to the differences in principal reinforcement bar diameter. In slabs with stirrups, the stirrups were spaced along the principal steel bar at the spacings shown in

Table 4.2, never encompassing the temperature steel.

The slabs were constructed in the laboratory with much care to ensure quality construction with minimal error in reinforcement placement. Figures 4.12 through 4.27 are photographs of slabs no. 1 and 16 prior to the placement of concrete. Figure 4.28 is a close-up view of the lacing in slab no. 9, and Figure 4.29 is a close-up view of the stirrups in slab no. 16.

4.3 Reaction Structure Details

Figure 4.30 shows a cross-sectional view of the reaction structure. The reaction structure had a removable door to allow access to the space beneath the slab specimen particularly for instrumentation requirements. Placement of a 36- by 24-inch slab in the reaction structure allowed 6 inches of the slab at each end to be clamped by a steel plate that was bolted into position, thereby leaving a 24- by 24-inch one-way restrained slab to be loaded with uniform pressure.

4.4 Instrumentation

Each slab was instrumented for strain, displacement, and pressure measurements. The data were digitally recorded with a personal computer. Two displacement transducers were used in each experiment to measure vertical displacement of the slab, one at one-quarter span and one at midspan. The displacement transducers used were Celesco Model PT-101, having a working range of 10 inches. These transducers measured the displacement of the slab by means of a potentiometer which detected the extension and retraction of a cable attached to a spring inside the transducer. More specifically, a Celesco Model PT-101 transducer contains a

springmotor that winds a cable around a drum that is attached to a linear rotary potentiometer. When the cable is completely retracted, the potentiometer is at one end of its range. As the cable is extended, the drum rotates (thus rotating the potentiometer) until the cable is at full extension and the potentiometer is at the other end of its range. A DC voltage is applied across the potentiometer, and the output is taken from the potentiometer's wiper. As the cable is retracted and the wiper moves along the potentiometer, the output voltage varies since the potentiometer acts as a voltage divider. The body of each transducer was mounted to the floor of the reaction structure, and the cable was attached to a hook glued to the slab surface. Retraction of the cables into the transducers' bodies occurred as the slab deflected and downward displacement occurred at the one-quarter span and midspan locations. Two single-axis, metal film, 0.125-inch-long, 350-ohm, strain gage pairs were installed on principal reinforcement in each slab. Each pair consisted of a strain gage on a top bar and one on a bottom bar directly below. One pair was located at midspan (ST-1, SB-1), and one was located at one-quarter span (ST-2, SB-2).

Strain gages were also installed at mid-height on shear steel in the slabs that contained shear reinforcement. Strain gages were placed on lacing bars in laced slabs at locations along the length of the slabs similar to the locations of stirrups with gages in the corresponding slabs with stirrups. The gages were placed on the shear reinforcement associated with the center principal steel bars. Figures 4.31 through 4.37 show the locations of the strain gages on the shear reinforcement in the

slabs. Two Kulite Model HKM-S375, 500-psi-range pressure gages (P1 and P2) were mounted in the bonnet of the test chamber in order to measure the water pressure applied to the slab.

4.5 Experimental Procedure

The 4-foot diameter blast load generator (Figure 4.38) was used to slowly load the slabs with water pressure. Huff (21) presented a detailed description of the test device, which is capable of developing static loads up to 500 psi. Preparations for the experiments began with the reaction structure being placed inside the test chamber and surrounded with compacted sand. A slab was then placed on the reaction structure. The wire leads from the instrumentation gages and transducers were connected. After placing the removable door into position, the sand backfill was completed on the door side of the reaction structure. A 1/8-inch-thick fiber-reinforced neoprene rubber membrane and a 1/8-inch-thick unreinforced neoprene rubber membrane were placed over the slab, and 1/2- by 6- by 24-inch steel plates were bolted into position at each support as shown in Figure 4.39. Prior to the bolting of the plates, a waterproofing putty was placed between the rubber membrane and the steel plates to seal gaps around the bolts in order to prevent a loss of water pressure during the experiment. A torque wrench was used to manually achieve approximately 50 foot-pounds on each bolt, and a consistent sequence of tightening the bolts was used for each experiment. The bonnet was bolted into position with forty 1-1/8-inch-diameter bolts tightened with a pneumatic wrench. A commercial waterline was diverted to the chamber's bonnet, and a

time of approximately 18 minutes was required to fill the bonnet volume of the chamber. A relief plug in the top of the bonnet indicated when the bonnet had been filled. At that time, the waterline valve was closed to allow closing of the relief plug. The waterline valve was again opened slowly, inducing a slowly increasing load to the slab's surface. A pneumatic water pump was connected to the waterline to facilitate water pressure loading in the case that commercial line pressure was not great enough to reach ultimate resistance of the slab in any of the experiments. Monitoring of the pressure gages and deflection gages indicated the behavior of the slab during the experiment and enabled this author to make a decision for termination by closing the waterline valve. The loading was controlled at a slowly changing rate, resulting in a load application time of several minutes. Following termination of the experiment, the bonnet was drained and removed. Detailed measurements and photographs of the slab were taken after removal of the neoprene membrane. Finally, the damaged slab was removed and the reaction structure was prepared for another slab.

4.6 Material Properties

The sixteen slabs were cast from one batch of concrete, which was proportioned to give a compressive strength of approximately 4,000 psi in about seven months. This time period was required since the project funding allowed casting of the slabs in the fall of 1990 and testing in the summer of 1991. Ten test cylinders were cast. Results of the uniaxial concrete cylinder tests are presented in Table 4.3.

D1, D2, and D3 deformed wires were used as reinforcement in the slabs. The wire was heat-treated in an oven at WES with the goal of producing a definite yield point at a yield stress of approximately 60,000 psi. Before heat treatment, the wire had an approximate yield stress of 90,000 psi. Numerous trials with various oven temperatures were required before satisfactory results were obtained. The results of tensile tests performed on specimens from heat-treated batches used in construction are presented in Table 4.4. Figure 4.40 is typical of the stress-strain curves plotted during the tensile tests.

Table 4.1 Slab Characteristics (Qualitative)

Slab	$\rho_{tension}$	ρ_{shear}	Lacing	Stirrups	Principal Steel Spacing	Shear Steel Spacing
1	small	none	-	-	0.67d	-
2	medium	none	-	-	0.63d	-
3	large	none	-	-	0.53d	-
4	small	small	x		0.67d	d
5	large	small	x		0.55d	d
6	small	medium	x		0.67d	3[illegible]
7	medium	medium	x		0.63d	3d/4
8	small	large	x		0.67d	d/2
9	large	large	x		0.55d	d/2
10	small	small		x	0.67d	d
11	small	medium		x	0.67d	3d/4
12	medium	medium		x	0.63d	3d/4
13	medium (Temperature steel placed exterior to principal steel)	medium		x	0.63d	3d/4
14	small	large		x	0.67d	d/2
15	large	small		x	0.55d	d
16	large	large		x	0.55d	d/2

Table 4.2 Slab Characteristics (Quantitative)

Slab	$\rho_{tension}$	ρ_{shear}	Lacing	Stirrups	Principal Steel Type* and Spacing (type/inches)	Shear Steel Spacing (inches)
1	0.0025	none	-	-	D1 / 1.60	-
2	0.0056	none	-	-	D2 / 1.50	-
3	0.0097	none	-	-	D3 / 1.33	-
4	0.0025	0.0026	x		D1 / 1.60	2.4
5	0.0097	0.0031	x		D3 / 1.33	2.4
6	0.0025	0.0034	x		D1 / 1.60	1.85
7	0.0056	0.0036	x		D2 / 1.50	1.85
8	0.0025	0.0052	x		D1 / 1.60	1.2
9	0.0097	0.0063	x		D3 / 1.33	1.2
10	0.0025	0.0026		x	D1 / 1.60	2.4
11	0.0025	0.0034		x	D1 / 1.60	1.85
12	0.0056	0.0036		x	D2 / 1.50	1.85
13	0.0056	0.0036		x	D2 / 1.50	1.85
	(Temperature steel placed exterior to principal steel)					
14	0.0025	0.0052		x	D1 / 1.60	1.2
15	0.0097	0.0031		x	D3 / 1.33	2.4
16	0.0097	0.0063		x	D3 / 1.33	1.2

* D1, D2, and D3 deformed wires have nominal cross-sectional areas of approximately 0.01, 0.02, and 0.03 inches, respectively.

Table 4.3 Results of Concrete Cylinder Tests

Cylinder	Age (days)	Compressive Strength (psi)
1	7	2780
2	7	2600
3	28	3400
4	28	3660
5	243	4400
6	243	4050
7	243	4260
8	243	4100
9	243	4120
10	243	3980

Table 4.4 Tensile Tests for Steel Reinforcement (deformed wire)

Wire Type*	Yield Stress (psi)	Ultimate Stress (psi)
D1	52,860	58,040
	52,680	58,040
	60,710	62,500
	58,040	62,050
	54,460	59,820
D2	62,500	72,320
	61,610	73,210
	67,860	77,230
	61,430	72,320
	55,360	65,180
D3	62,280	71,170
	64,290	72,770
	66,070	72,770
	64,730	71,880
	66,070	73,210

* D1, D2, and D3 deformed wires have nominal cross-sectional areas of approximately 0.01, 0.02, and 0.03 inches, respectively.

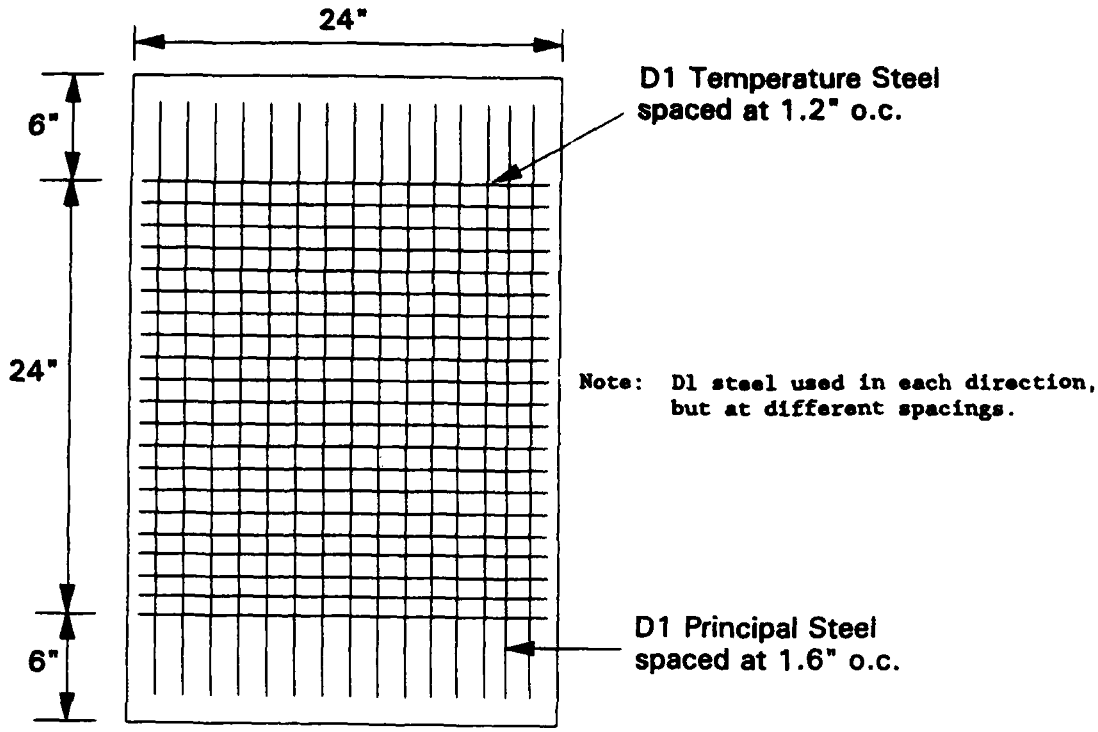


Figure 4.1. Plan View of Slab Nos. 1, 4, 6, 8, 10, 11, and 14

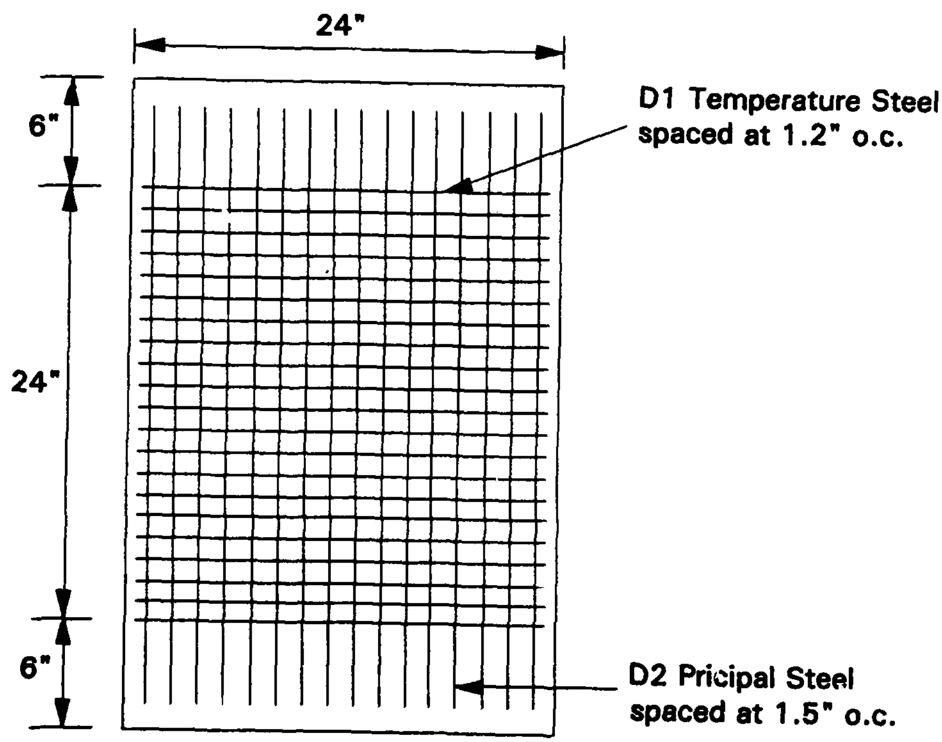


Figure 4.2. Plan View of Slab Nos. 2, 7, 12, and 13

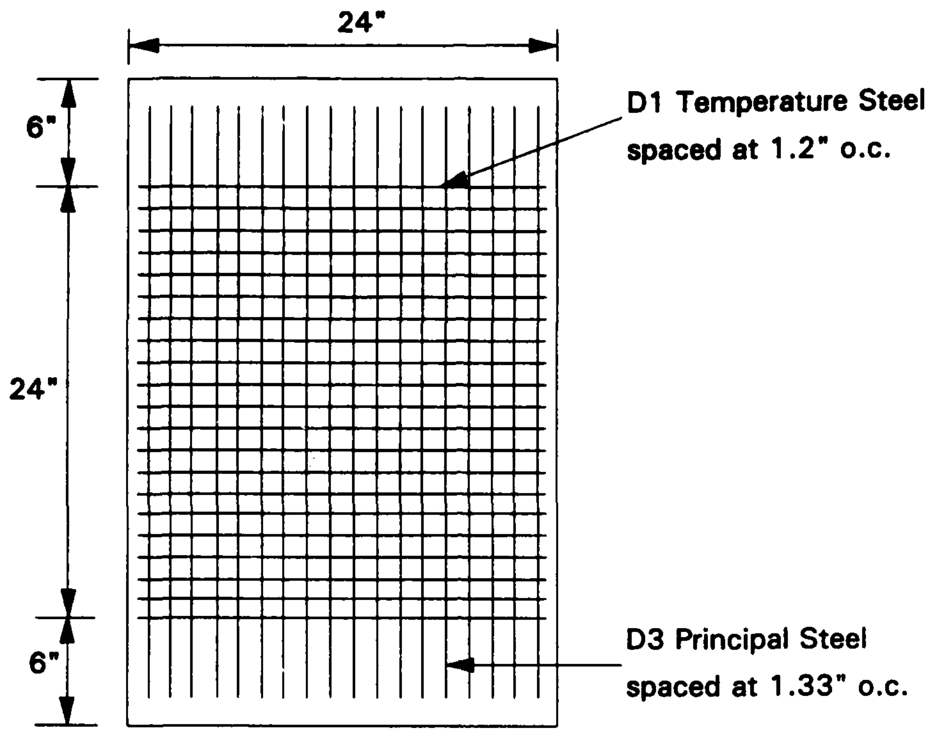


Figure 4.3. Plan View of Slab Nos. 3, 5, 9, 15, and 16

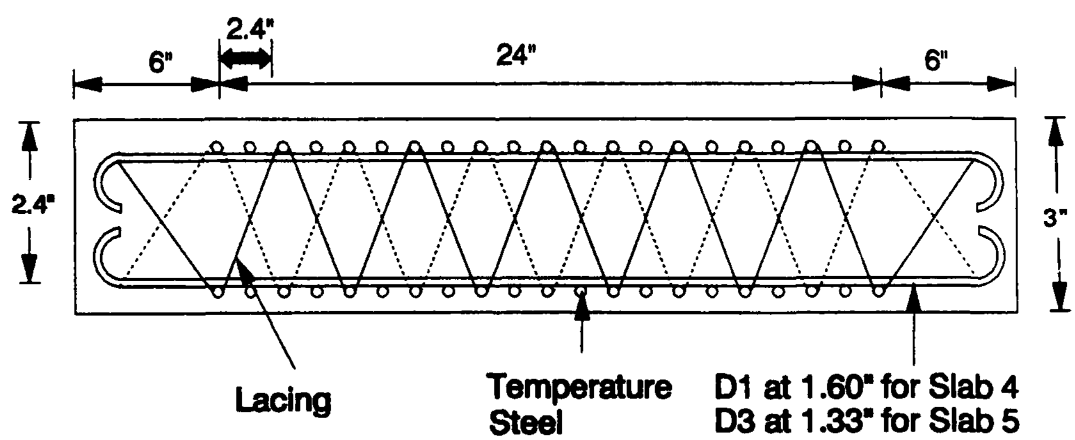


Figure 4.4. Sectional View Through Length of Slab Nos. 4 and 5

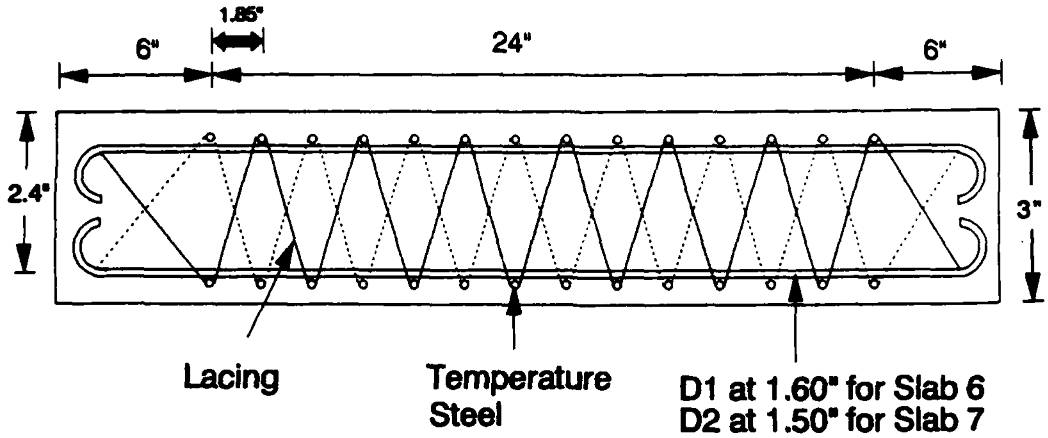


Figure 4.5. Sectional View Through Length of Slab Nos. 6 and 7

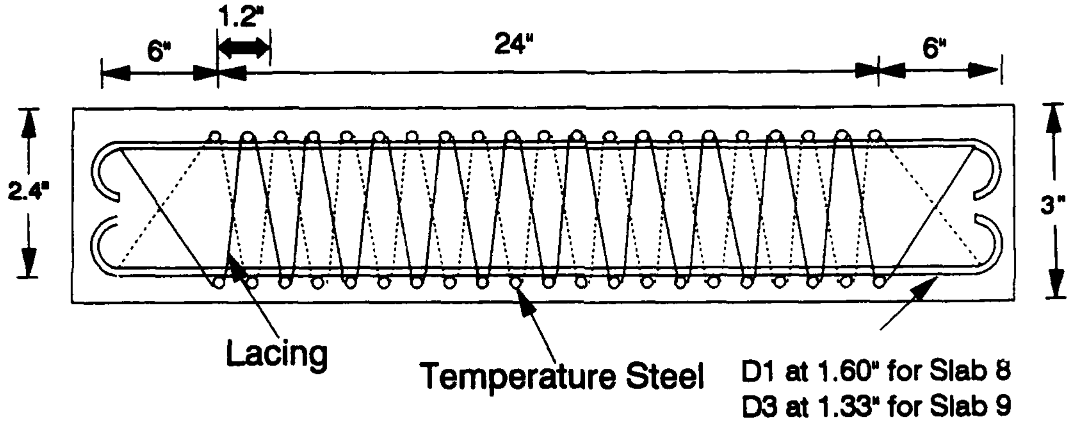


Figure 4.6. Sectional View Through Length of Slab Nos. 8 and 9

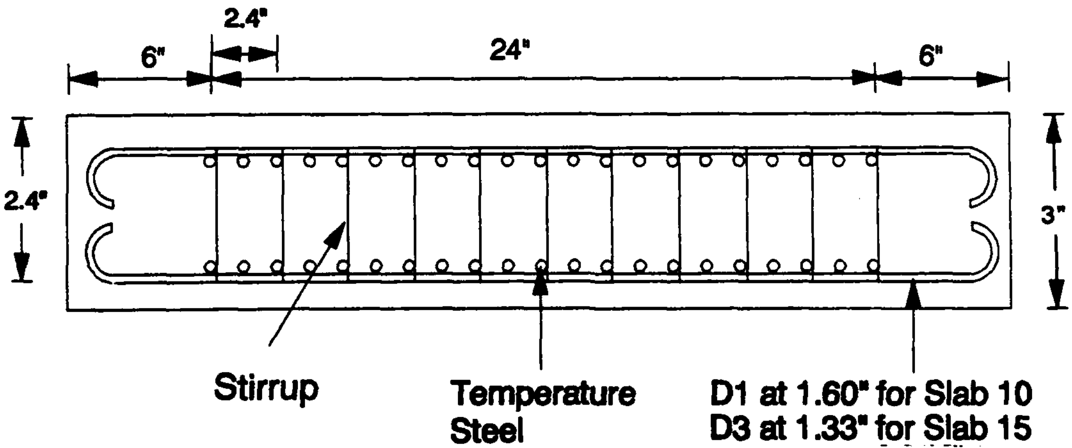


Figure 4.7. Sectional View Through Length of Slab Nos. 10 and 15

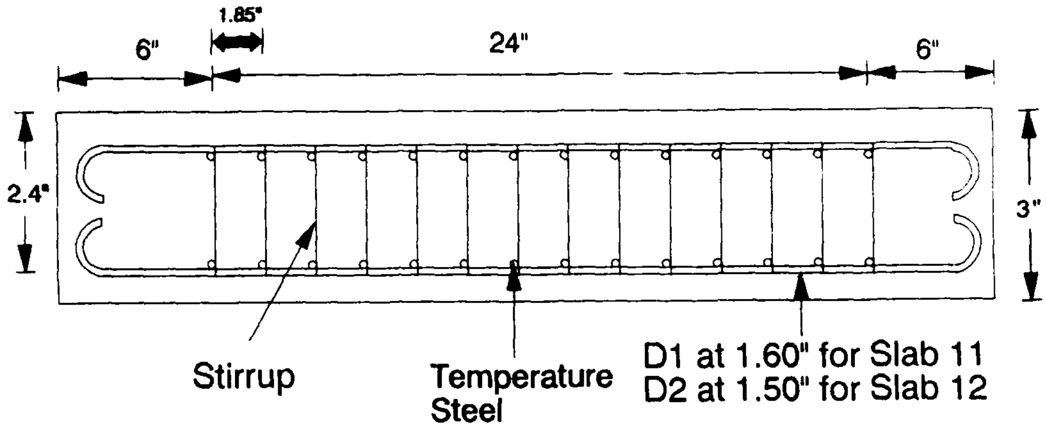


Figure 4.8. Sectional View Through Length of Slab Nos. 11 and 12

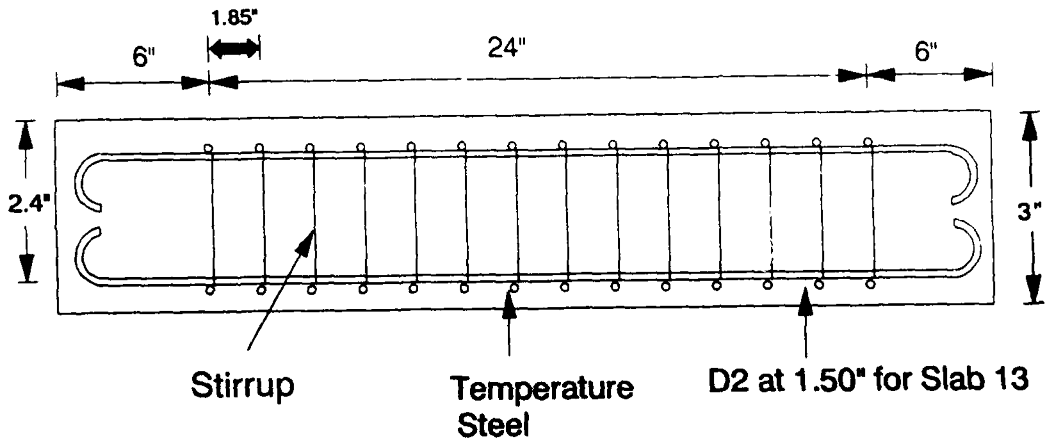


Figure 4.9. Sectional View Through Length of Slab No. 13

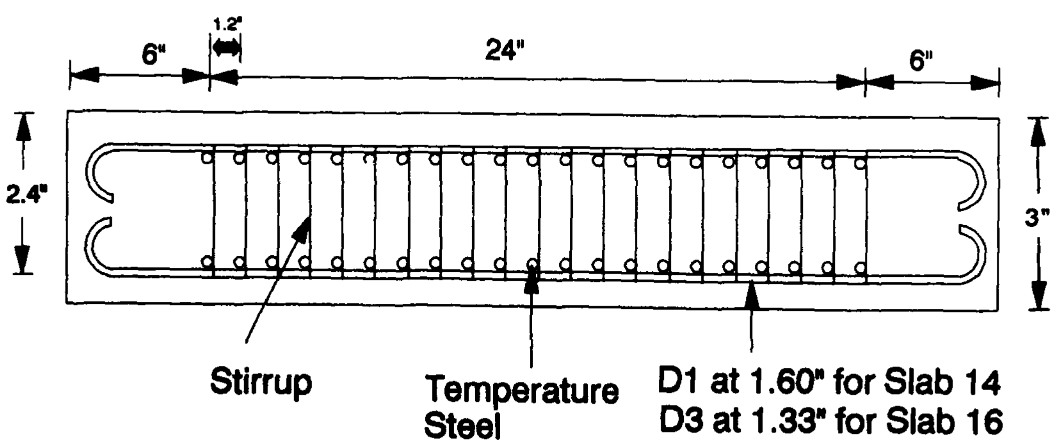


Figure 4.10. Sectional View Through Length of Slab Nos. 14 and 16

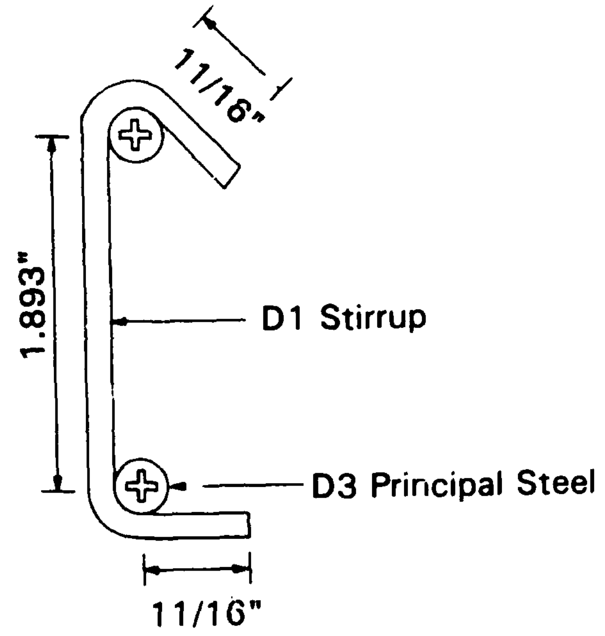


Figure 4.11. Stirrup Details for Slabs with D3 Principal Steel

Figure 4.12. Slab No. 1 Prior to Concrete Placement

Figure 4.13. Slab No. 2 Prior to Concrete Placement

Figure 4.14. Slab No. 3 Prior to Concrete Placement

Figure 4.15. Slab No. 4 Prior to Concrete Placement

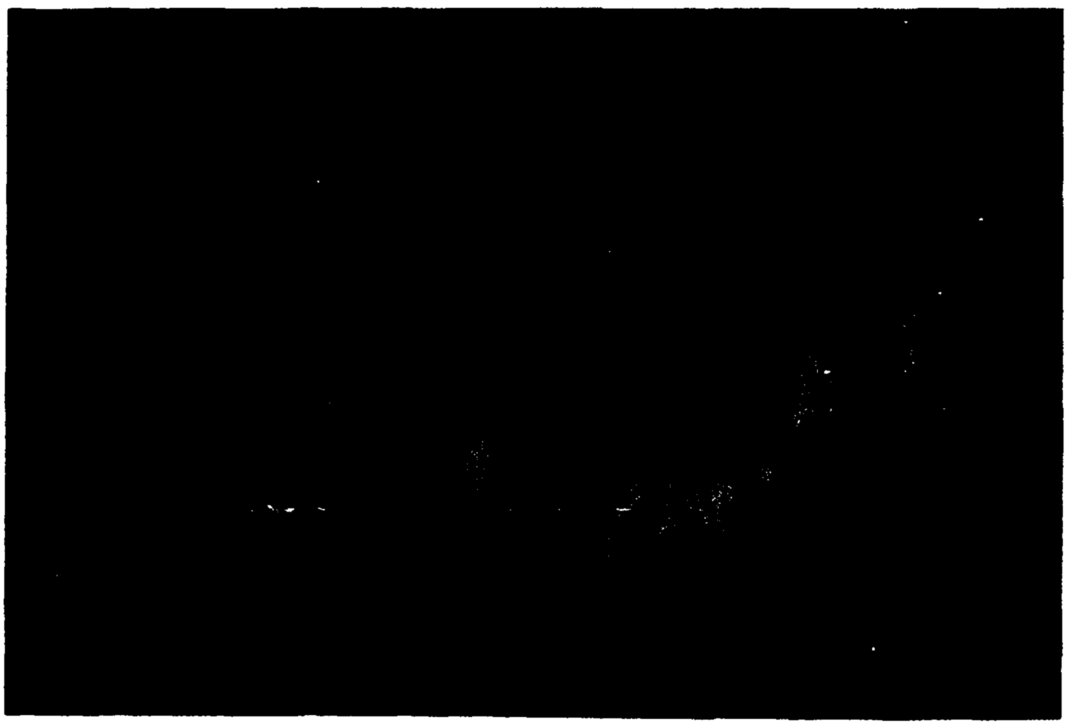


Figure 4.16. Slab No. 5 Prior to Concrete Placement

Figure 4.17. Slab No. 6 Prior to Concrete Placement

Figure 4.18. Slab No. 7 Prior to Concrete Placement

Figure 4.19. Slab No. 8 Prior to Concrete Placement

Fiaure 4.20. Slab No. 9 Prior to Concrete Placement

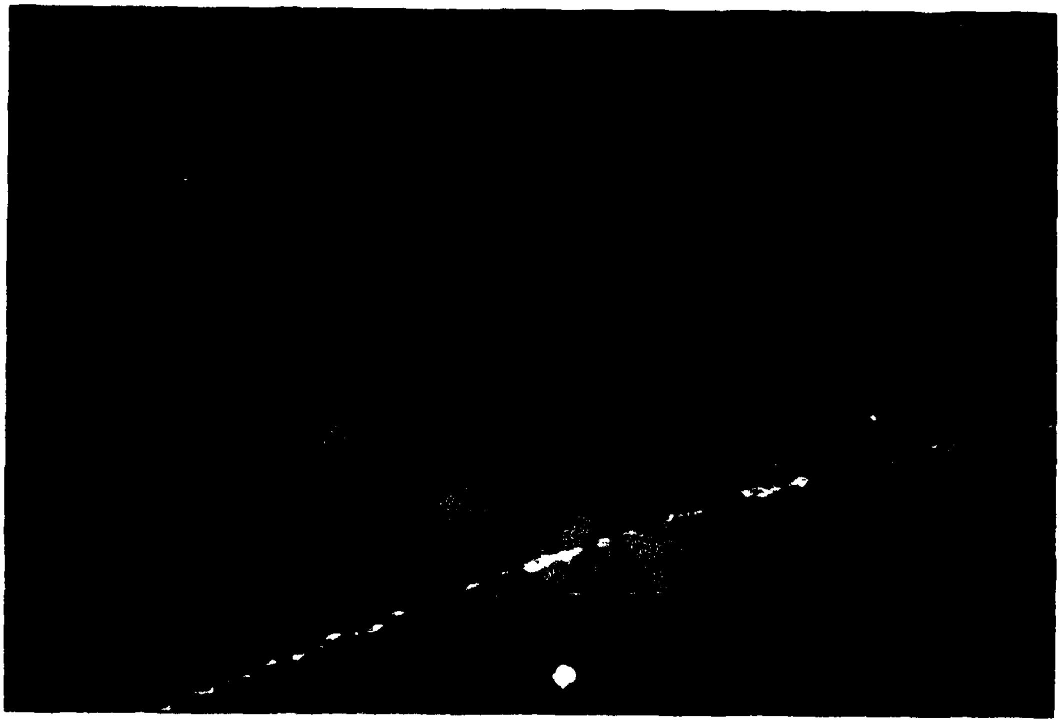


Figure 4.21. Slab No. 10 Prior to Concrete Placement

Figure 4.22. Slab No. 11 Prior to Concrete Placement

Figure 4.23. Slab No. 12 Prior to Concrete Placement

Figure 4.24. Slab No. 13 Prior to Concrete Placement

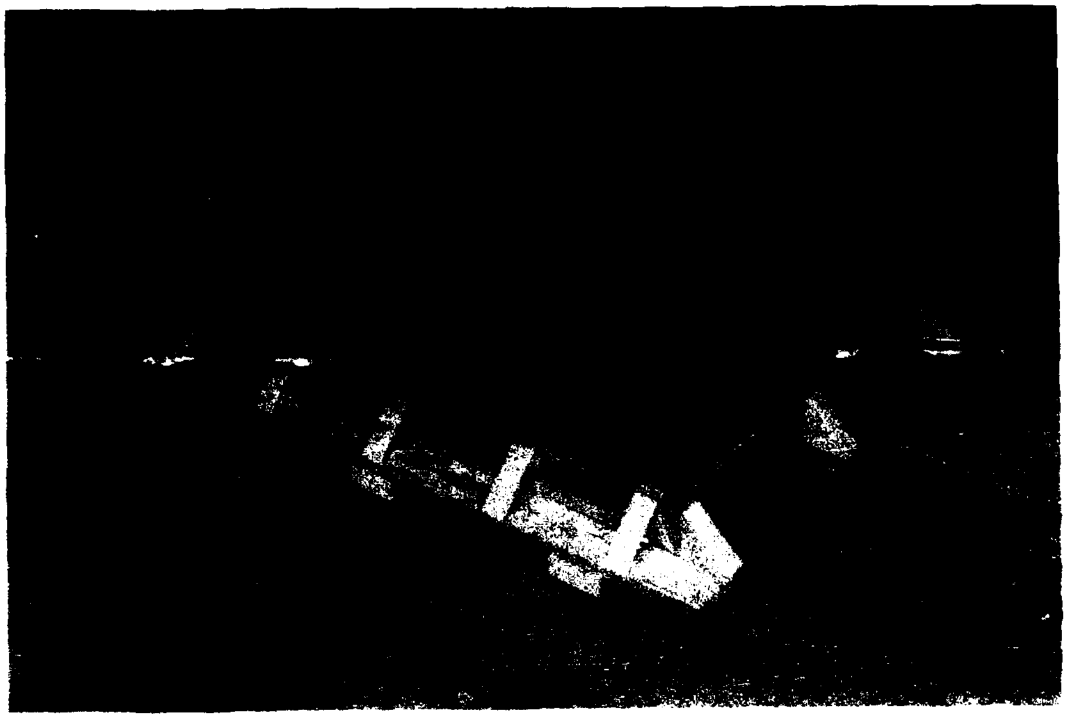


Figure 4.25. Slab No. 14 Prior to Concrete Placement

Figure 4.26. Slab No. 15 Prior to Concrete Placement

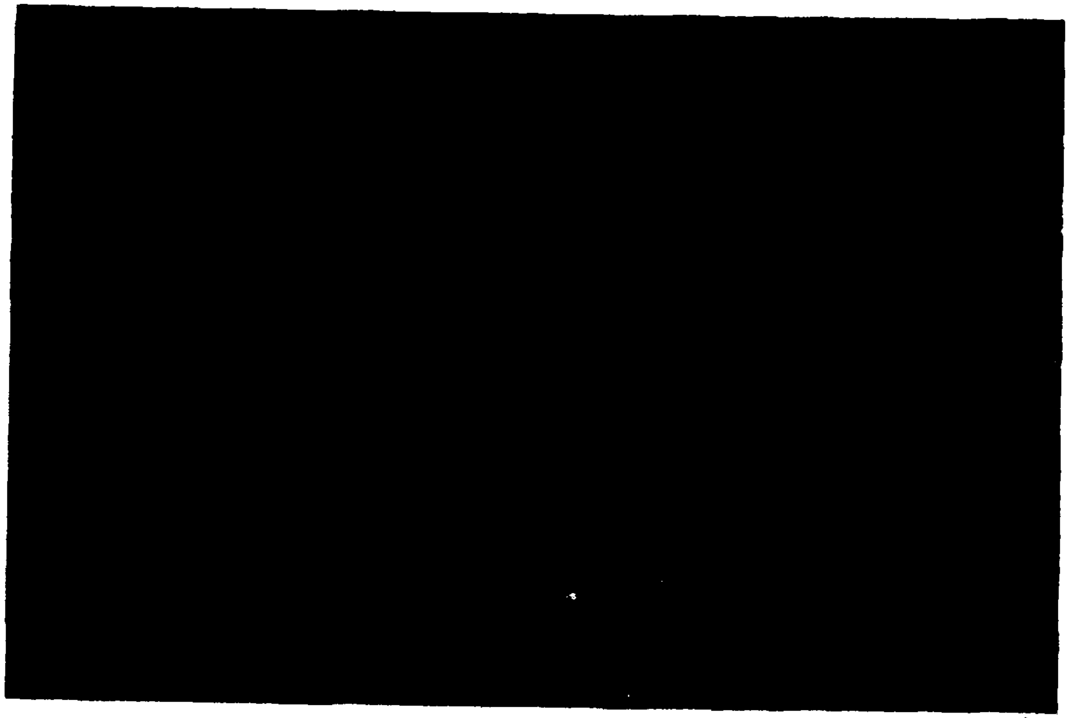


Figure 4.27. Slab No. 16 Prior to Concrete Placement

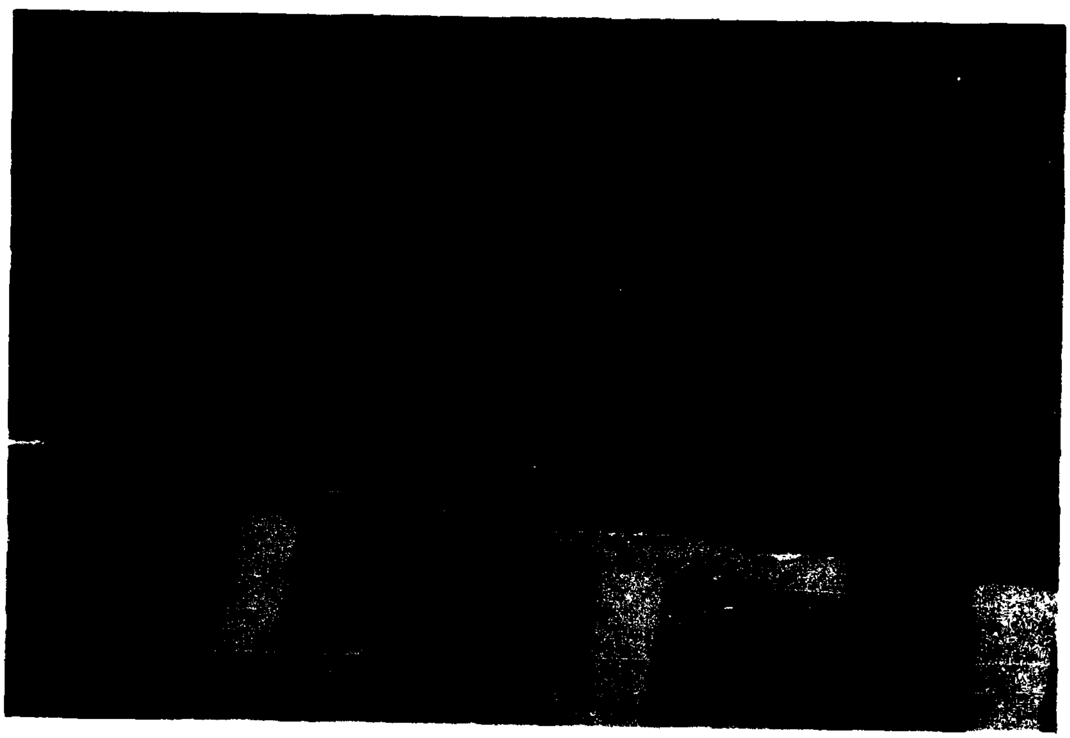


Figure 4.28. Close-up View of Lacing in Slab No. 9

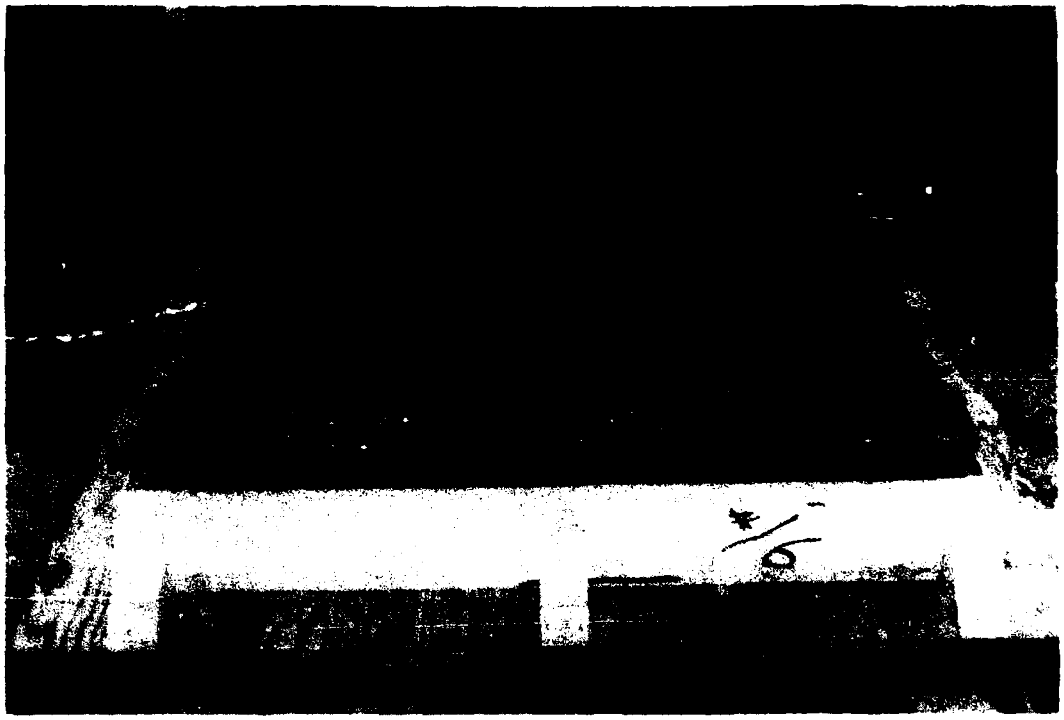


Figure 4.29. Close-up View of Stirrups in Slab No. 16

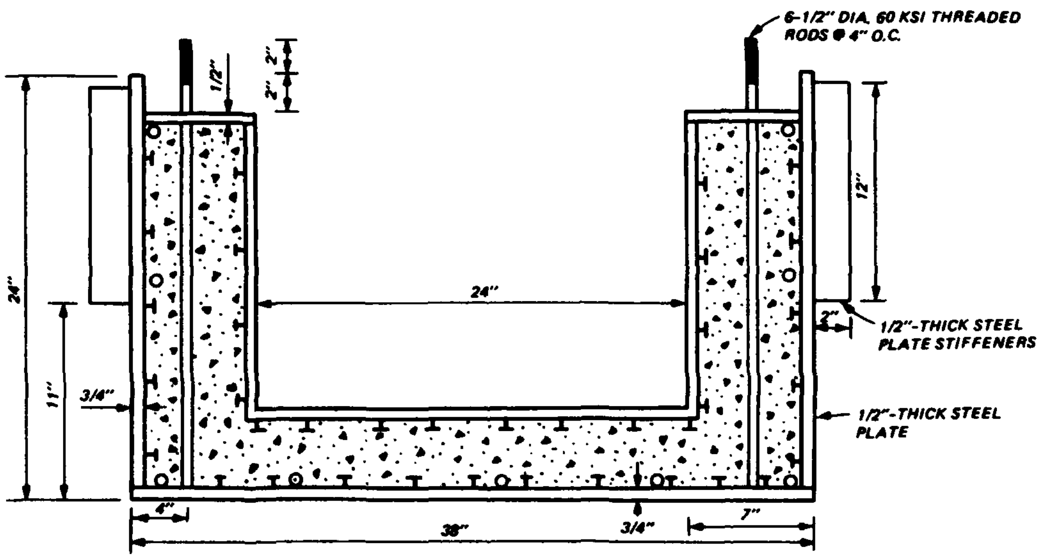


Figure 4.30. Cross Section of Reaction Structure

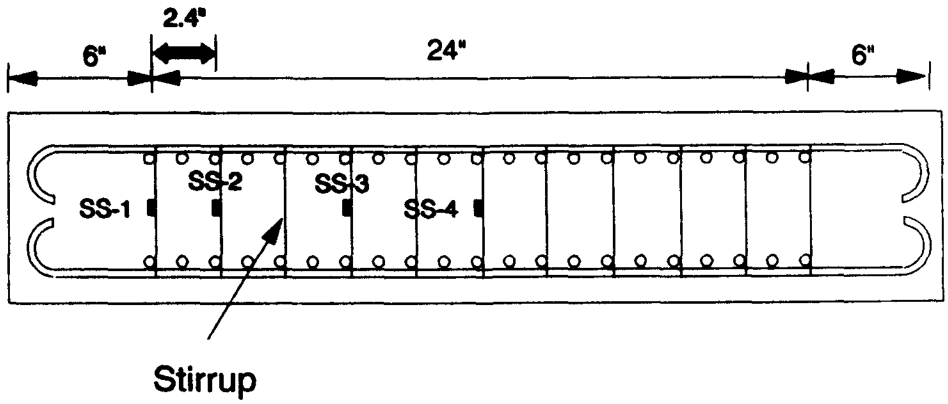


Figure 4.31. Strain Gage Locations on Stirrups in Slab Nos. 10 and 15

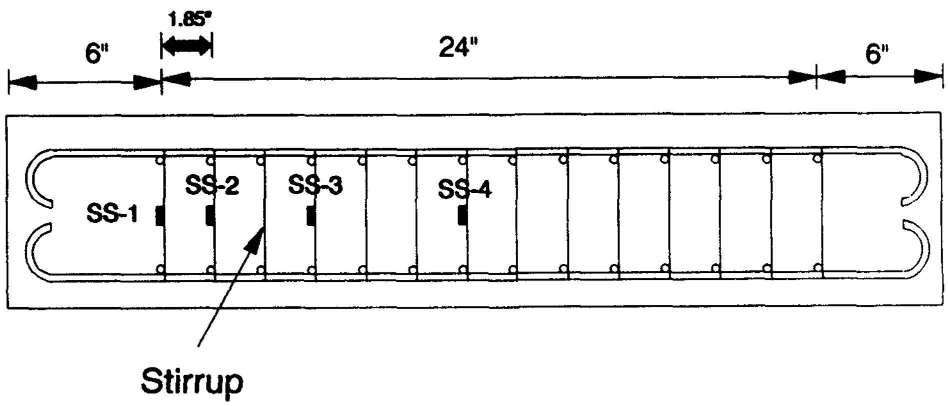


Figure 4.32. Strain Gage Locations on Stirrups in Slab Nos. 11 and 12

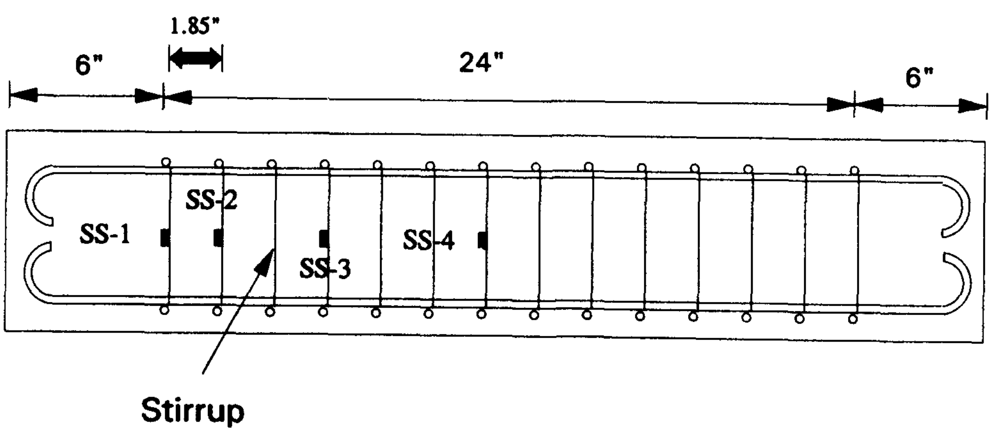


Figure 4.33. Strain Gage Locations on Stirrups in Slab No. 13

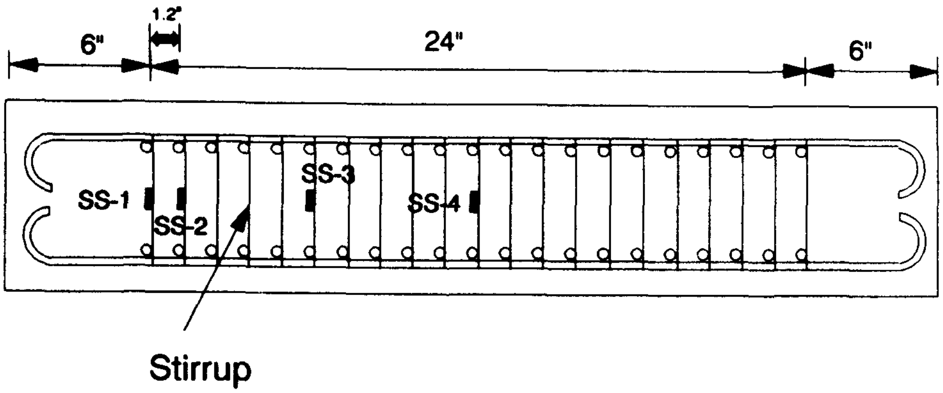


Figure 4.34. Strain Gage Locations on Stirrups in Slab Nos. 14 and 16

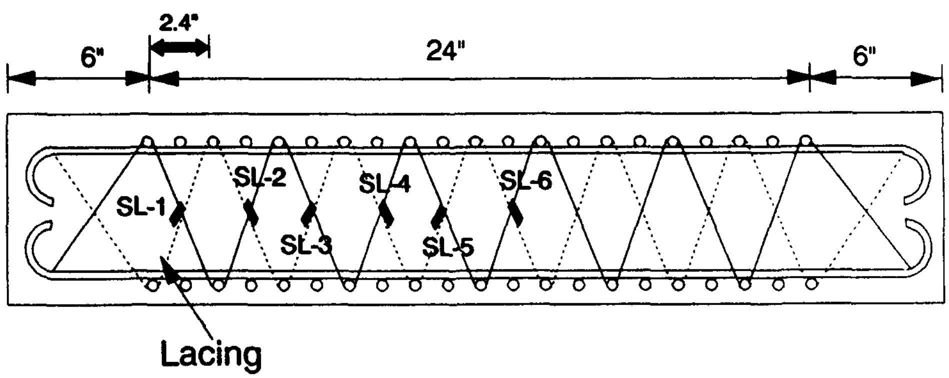


Figure 4.35. Strain Gage Locations on Lacing in Slab Nos. 4 and 5

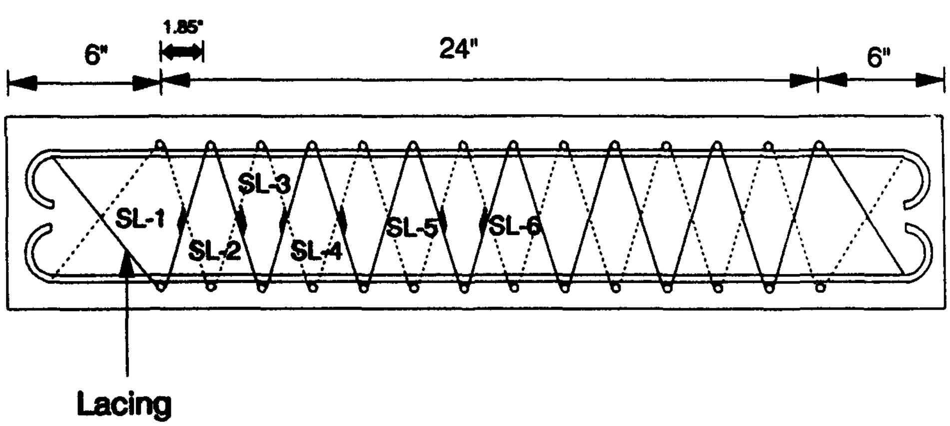


Figure 4.36. Strain Gage Locations on Lacing in Slab Nos. 6 and 7

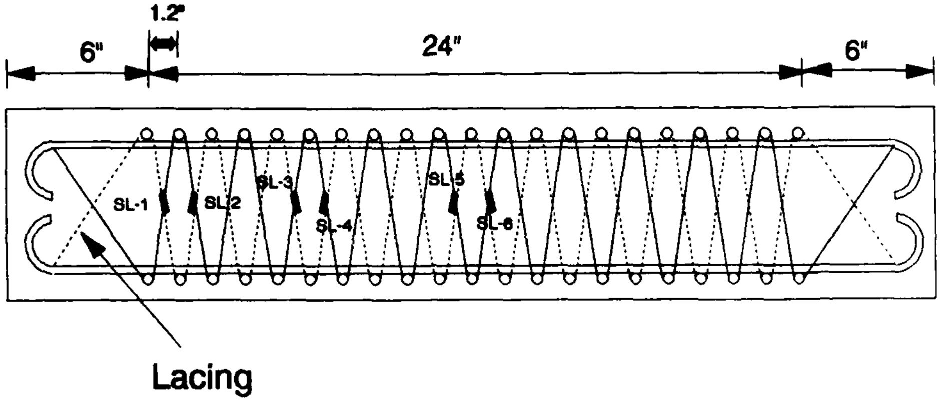


Figure 4.37. Strain Gage Locations on Lacing in Slab Nos. 8 and 9

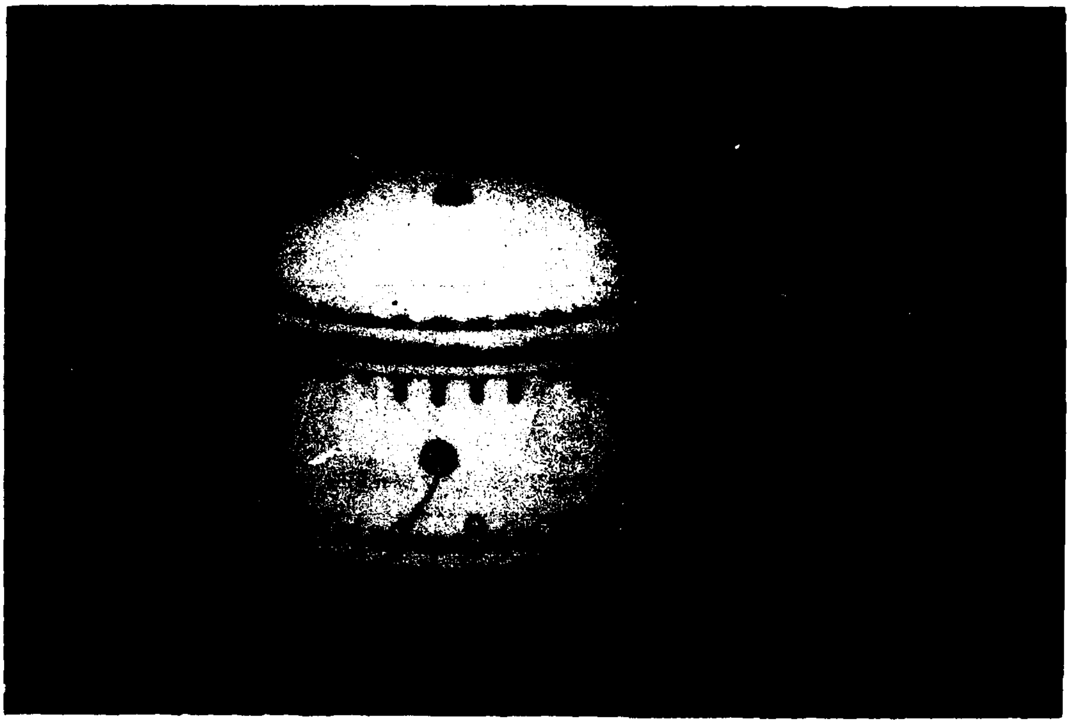


Figure 4.38. Four-Foot-Diameter Blast Load Generator

Figure 4.39. Membrane with Steel Plates in-place

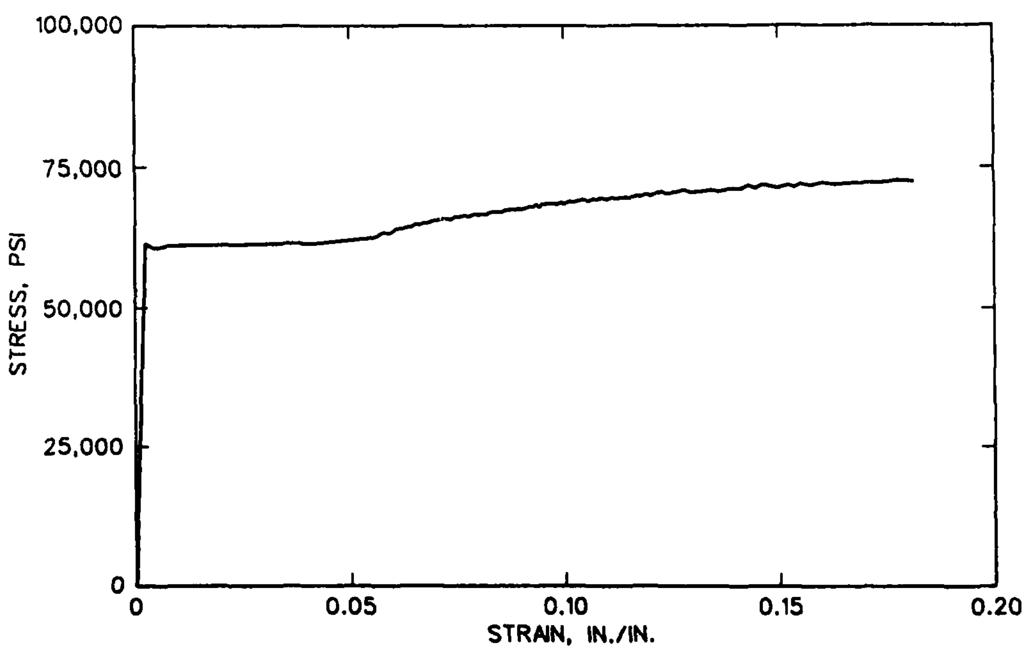


Figure 4.40. Typical Stress-Strain Curve for Reinforcement

CHAPTER 5

RESULTS AND ANALYSIS

5.1 Introduction

The electronically recorded data plots for all sixteen experiments are presented in the Appendix. In this chapter, the data are presented in various forms (i.e., tables and composite plots), and the results of the experiments are evaluated. Posttest photographs and sketches showing the extent of structural damage are included. Section 5.2 provides an overall presentation and discussion of the experimental results. Sections 5.3 and 5.4 respectively address two specific regions of response: ultimate capacity and tension membrane. The implications of the results on design criteria (allowable response limits) are expressed in Section 5.5.

5.2 Results and Discussion

Figures 5.1 through 5.16 show the posttest condition of each slab immediately after removal of the neoprene membrane, and Figure 5.17 is a posttest view of the undersurfaces of all sixteen slabs. In Figure 5.17, the slabs were placed in increasing order from left to right with slabs no. 1 through no. 5 being shown in the front row. In general, each slab responded as a three-hinged mechanism. The posttest-measured midspan deflection (Δ) of each slab, along with values of two parameters often used to quantify the response or indicate the ductility of a slab, are presented in Table 5.1. The ratio of midspan deflection (measured posttest) to clear span length is given for each slab. Also, the maximum support rotation for each slab is given. Consistent with TM 5-

1300 (1), these support rotation values are computed by simply taking the arctan of the quotient of the midspan deflection divided by one-half the clear span length.

The discussion and analyses presented in this chapter greatly rely on the slabs' load-deflection data. Figure 5.18 shows the general shape of the midspan load-deflection curves. Although plots of the load-deflection curves for the slabs will be presented in subsequent sections, values of load and deflection at points A through D of Figure 5.18 are given in Table 5.2 for convenience in numerical comparisons. The decision to terminate an experiment depended upon the trend of the monitored load-deflection curves or was due to a water leak causing pressure loss; therefore, the deflection at termination varied among the slabs. The complete load-deflection curves at midspan were not recorded for slabs no. 12, 14, and 16 due to degradation of the deflection gage connections to the slabs. Large cracks, as presented in Table 5.3, formed directly at the points of connection during the experiments and dislodged the deflection gage cable at midspan for these slabs. However, the complete load-deflection curves measured at the one-quarter span location were successfully recorded and will be discussed.

To complement the discussion related to deflection data, deflection profiles for slabs no. 1 through 16 are shown in Figures 5.19 through 5.34, respectively. The profiles are approximations developed by connecting the support (zero deflection) with the quarterspan and midspan deflections. The top portion of each figure shows the deflection profile as the load increases up to the ultimate capacity (termed "peak" in the

legends of the figures). The lower portion of each figure shows the deflection profile as the load decreases in the region corresponding to that from point "A" to between points "B" and "C" in Figure 5.18. The profiles corresponding to the region following the ultimate load are not given for slab no. 16 since the deflection gage became unattached from the slab prior to the attainment of the ultimate capacity. For all slabs, the deflection profiles are not presented for midspan deflections greater than 3.0 inches nor greater than that corresponding to point "C" of Figure 5.18 because the midspan deflection measurements were not accurate at greater deflections. These inaccuracies were due to the significant geometry changes of the slabs and are somewhat quantified in Table 5.4. The posttest measured deflection was greater than the electronically recorded maximum deflection for each slab. The primary reason for the discrepancies between the posttest measured deflections and the electronically recorded maximum deflections was the change in the geometry of each slab during the experiments. As a slab deflected (in the form of a three-hinge mechanism), a prominent crack formed at midspan. In most cases, the crack formed slightly to the left or to the right of the point of connection of the deflection gage to the slab. As slab deflection (or rotation at the hinge lines) continued, the deflection gage point of connection was moved both horizontally and vertically. The horizontal component tended to pull the cable of the deflection gage out of the gage housing as opposed to the desired retraction of the cable into the gage housing. Therefore, error was introduced into the recorded deflection values, particularly at large deflections. The R/M

ratio, defined and given in Table 5.4, is an indication of the discrepancy in recorded and measured deflections at midspan. The value of discrepancy was lowest for slabs no. 2 and 3, which included shear response and were not pushed to very large deflections.

Figures 5.19 through 5.34 present deflection profiles for the slabs at various load levels. The legends in the figures include notations, such as ST-2, indicating that the particular strain gage reached yield strain at that load. Strain gages that did not indicate yielding of reinforcement are not given in the figures. ST-1 and SB-1 were strain gages respectively located on the top and bottom principal steel bars at the quarterspan of each slab. ST-2 and SB-2 were respectively located on the top and bottom principal steel bars at the midspan of each slab. The locations of strain gages on the shear reinforcement were presented in Chapter 4, specifically in Figures 4.31 through 4.37. It is not beneficial to attempt detailed comparisons of strain gage data for the slabs since localized yielding of a bar may have occurred at a location other than the gage location. For example, a strain gage located at midspan may not have indicated that yielding or rupture of a bar occurred even though it was obvious from posttest inspection that the bar ruptured. Therefore, the data should only be used to gain some insight into the general behavior of the slabs.

Slabs No. 1, 2, and 3

For discussion, the slabs may be grouped by design parameter values. Slabs no. 1, 2, and 3 were all constructed without shear

reinforcement and served as baseline slabs for this study. The load-response behavior and failure modes of these three slabs varied. The principal reinforcement ratio was small (0.0025) in slab no. 1, and flexural failure occurred prior to shear failure. The experiment was terminated when it appeared that the load resistance of the slab was rapidly deteriorating and that collapse was impending. This resulted in the well-defined three-hinge mechanism shown in the photograph of Figure 5.1.

The lack of shear reinforcement and the presence of a medium (0.0056) principal reinforcement ratio in slab no. 2 resulted in a combined flexure-shear failure mode as shown in Figure 5.2. Actually, the experiment on slab no. 2 was terminated due to a loss in the water pressure that composed the loading. This water leak occurred at one of the supports due to improper sealing around the bolts. Because of the failure mode, it was decided that slab no. 2 should not be reloaded.

As shown in Figure 5.3, shear was the dominate failure mode for slab no. 3, which possessed a large (0.0097) principal reinforcement ratio. The three different failure modes of slabs no. 1, 2, and 3 confirmed the need for these three baseline experiments within the shear reinforcement study.

Figures 5.19, 5.20, and 5.21 indicate that slabs no. 1, 2, and 3 responded in generally similar modes up to the peak load resistance (typically the ultimate flexural capacity, except for slab no. 3). These three figures show that some yielding of principal reinforcement occurred before the slabs attained the peak resistance. The deflection profiles indicate that the slab sections between yield lines did not remain perfectly straight and

undeformed.

Slabs No. 4 and 10

Slabs no. 4 and 10 differed only in the types of shear reinforcement (lacing in slab no. 4 and stirrups in slab no. 10). Figures 5.4 and 5.10 show that both slabs responded as well-defined three-hinge mechanisms, as was the case for the baseline slab (slab no. 1) corresponding to these two slabs. The posttest measurements presented in Table 5.1 indicated that slab no. 4 was pushed slightly further than slab no. 10 before experiment termination. Both slabs sustained support rotations beyond 22 degrees. Figures 5.35. 5.38, and 5.44 show the extent of concrete damage for slabs no. 1, 4, and 10, respectively. The region of particular interest is the hinge line at midspan on the bottom surface. While these figures indicate some variation in the extent of medium and light damage, the extent of heavy damage was very similar for each slab. The values in Table 5.3 indicate that the widths of the bottom cracks at midspan were also similar for slabs no. 4 and 10. Variations in these crack width values were partly due to the maximum midspan deflections of the slabs since the bottom surface midspan crack of a slab that deforms as a three-hinge mechanism opens as the slab geometry changes with increasing deflection. For example, the crack widths for slabs no. 1, 4, and 10 were approximately 1.88, 2.5, and 2.13 inches, respectively. The midspan deflections (given in Table 5.4) for slabs no. 1, 4, and 10 were 4.4, 5.5, and 5.0 inches, respectively. The variations in the extent of light and medium damage in slabs no. 1, 4, and 10 may lead one to believe that the

lacing in slab no. 4 caused the damage to be concentrated at the hinge line; however, such a conclusion should not be made until similar comparisons are evaluated for all slabs in this study. Actually, "medium" and "light" damage levels in Figures 5.35 through 5.50 refer to regions of small and hairline cracks, respectively. Therefore, both the medium and light damage levels given in these figures correspond to significantly less damage than the heavy damage level. In fact, very few of the cracks associated with the medium and light damage levels are visible in the photograph presented in Figure 5.17.

Figures 5.22 and 5.28 indicate that the shapes of the deflection profiles for slabs no. 4 and 10 were similar up to near the deflection at which tensile membrane behavior begins. To a significantly greater extent than occurred for slab no. 1, the sections between yield lines straightened as deflections increased and deformation at the hinge lines became more pronounced. Figures 5.22 and 5.28 indicate that principal reinforcement yielded prior to the slabs reaching their ultimate capacities. Also, the strain gage data indicated yielding of the lacing bars at midspan in slab no. 4 and yielding of stirrups at midspan in slab no. 10 prior to the slabs reaching their ultimate capacities. The data indicated that yielding of the lacing at midspan in slab no. 4 occurred earlier (lower load and deflection) than did the yielding of stirrups in slab no. 10.

<u>Slabs No. 6 and 11</u>

Similar to slabs no. 4 and 10, slabs no. 6 and 11 differed only in the types of shear reinforcement (slab no. 1 was the

baseline), but the amount of shear reinforcement was greater than in the case of slabs no. 4 and 10. Support rotations of approximately 25 and 26 degrees were sustained for slabs no. 6 and 11, respectively. As shown in Figures 5.6 and 5.11, both slabs responded as well-defined three-hinge mechanisms. Figures 5.40 and 5.45 indicate that the damage at midspan on the bottom surface was more concentrated near the midspan hinge for slab no. 11 (contained stirrups) than for slab no. 6 (contained lacing). This is the reverse of the observation for the effects of stirrups and lacing in slabs no. 4 and 10. Dominate midspan crack widths were very similar for slabs no. 6 and 11 at values of approximately 3.5 and 3.4 inches, respectively.

The deflection profiles presented in Figure 5.29 for slab no. 11 resemble the shapes of the profiles for slabs no. 4 and 10. However, the deflection profiles given in Figure 5.24 for slab no. 6 are different from profiles discussed thus far in that they indicate a steeper slope for the slab section between the quarterspan and midspan points than for the section between the support and quarterspan. Such a deflection profile is representative of a cantilever; however, the plastic hinges at the supports can not provide the moment capacity required to induce the cantilever-type profile. As shown in Figure 5.6, there was no visual evidence of negative bending between the hinge lines. Therefore, the deflection profiles for slab no. 6 indicate error in the deflection measurements. The error in the deflection measurements may have been the result of deterioration of the bottom surface of the slab at midspan. This deterioration included cracking and some scabbing of the concrete cover. Since

the cable (in tension) of the deflection gage was connected to the bottom surface of the slab, the deterioration of the surface resulted in midspan deflection measurements that were greater than the true deflections as the tension in the cable tended to pull the scabbed concrete from the slab. Although it has been stated that geometry changes at very large deflections tended to make the measurements be less than the true deflections, Figures 5.19 through 5.34 do not include the range of very large deflections where the effects of the geometry changes are significant. Therefore, the profiles that indicate negative bending between hinge lines should be assumed to be nearly straight lines. Figures 5.24 and 5.29 indicate that the yielding of principal reinforcement and shear reinforcement near the midspan of slabs no. 6 and 11 occurred prior to the slabs reaching their ultimate capacities. The data indicated that the lacing in slab no. 6 yielded at lower loads and deflections than did the stirrups in slab no. 11.

Slabs No. 8 and 14

Slabs no. 8 and 14 represented a further increase in the amount of shear reinforcement for the slabs with the small principal reinforcement ratio. Both of these slabs sustained support rotations of approximately 25 degrees. Slab no. 14 was one of the three slabs in which the midspan deflection gage became disconnected during the experiment; however, Figure 5.51 and the values in Table 5.5 indicate that the response of the two slabs were very similar throughout the entire range of deflections as based on the data measured at the one-quarter span. Figures 5.42

and 5.48 present the damage surveys for slabs no. 8 and 14, respectively. These figures indicate that both slabs incurred some spreading of medium and light damage at midspan; however, the region of heavy damage was slightly wider for slab no. 14. This observation indicates that the lacing in slab no. 8 allowed the concentration of cracking at the hinge line. The values in Table 5.3 indicate that the dominate midspan crack widths were similar, being slightly greater for slab no. 14 which also incurred slightly more deflection.

The deflection profiles for slabs no. 8 and 14, respectively shown in Figures 5.26 and 5.32, indicate yielding of principal reinforcement prior to the slabs attaining their ultimate capacities. The data also indicated that lacing yielded at the midspan of slab no. 8 prior to it reaching its ultimate capacity. Strain gages located on stirrups near the midspan of slab no. 14 did not indicate yielding of the stirrups. However, yielding of stirrups near the support apparently did occur for slab no. 14.

Slabs No. 7, 12, and 13

Only the medium category of shear reinforcement was investigated for the medium amount of principal reinforcement (baseline was slab no. 2). Slabs no. 7 and 12 differed only in the type of shear reinforcement. Slabs no. 7 (contained lacing) and 12 (contained stirrups) were pushed to support rotations of approximately 21 and 25 degrees, respectively. Figure 5.52 and the values in Table 5.5 indicate that the two slabs behaved very similarly, primarily differing in the maximum deflection attained. From Figures 5.7 and 5.12, it is obvious that the medium category

of shear reinforcement was sufficient to prevent shear failure (as opposed to the flexure\shear failure in the baseline slab no. 2). Also, the photographs indicate a smoothing of the midspan crack region of the three-hinge mechanism when compared to the previously discussed slabs that contained the small amount of principal reinforcement. Figures 5.41 and 5.46 indicate that the extent of heavy damage was greater for slab no. 12 than for slab no. 7. This is also reflected by the approximately 4.0-inch wide midspan crack incurred by slab no. 12 compared to the approximately 2.1-inch wide crack of slab no. 7 (given in Table 5.3). These differences in damage are reasonable when the maximum midspan deflection values (approximately 4.5 and 5.7 inches for slabs no. 7 and 12, respectively) are considered.

Slab no. 13 was similar to slab no. 12, differing only in the placement of the temperature reinforcement. The temperature reinforcement was placed exterior to the principal reinforcement in slab no. 13, but interior to the principal reinforcement in slab no. 12. This construction variation was included in the study in order to evaluate the differences in the temperature steel placement in slabs with lacing and stirrups. A previous study (10) indicated that the exterior placement of the temperature reinforcement may enhance the ductility of a slab, possibly overshadowing some effects of shear reinforcement. Although slab no. 13 was pushed significantly further than slab no. 12, its mode of response was not significantly different. The maximum load resistances attained for the secondary resistance (given as P_D in Table 5.2) were similar for slabs no. 12 and 13 at values of 43 and 41 psi, respectively. However, slab no. 13 was

capable of maintaining the peak reserve capacity up to a larger deflection as is evident from the values given in Table 5.1. Figure 5.13 shows a significant loss of concrete in the compressive crushing zone at midspan. The bending of the principal reinforcement in the midspan zone resembled that of slab no. 12. The large support rotation of approximately 30 degrees for slab no. 13 resulted in the concrete falling from the reinforcement. A small core of concrete remained attached to the reinforcement, primarily due to the shear reinforcement that was present in the form of stirrups. Figures 5.46 and 5.47 indicate that the spread of damage levels was similar for slabs no. 12 and 13. However, the crack width for slab no. 13 was significantly greater than that for slab no. 12 due to the greater midspan deflection as reflected by the posttest measured deflection of 7.0 inches (compared to 5.7 inches for slab no. 12).

Figures 5.25, 5.30, and 5.31 indicate that the deflection profiles of slabs no. 7, 12, and 13 were similar. Sections between hinge lines were generally straight, and yielding of principal reinforcement occurred prior to the slabs reaching their respective ultimate capacities. Although the strain gages on the lacing bars near midspan of slab no. 7 did not indicate yielding, gage SL-4 (located between quarterspan and midspan) did indicate yielding of the lacing bar. None of the strain gages on stirrups in slabs no. 12 and 13 indicated yielding of the stirrups.

Slabs No. 5 and 15

Slabs no. 5 and 15 each contained a small amount of shear reinforcement in the form of lacing bars and stirrups,

respectively. These slabs contained a large amount of principal reinforcement (the baseline slab was slab no. 3). Although a water leak caused termination of the experiment at a support rotation of approximately 24 degrees for slab no. 15, Figures 5.5 and 5.15 indicate that the failure modes for the two slabs were similar. Slab no. 5 was pushed to a support rotation of approximately 30 degrees. Although only a small amount of shear reinforcement was used in these slabs, the failure mode was primarily that of flexure rather than the shear failure that occurred in the baseline slab no. 3.

Figures 5.39 and 5.49 show that damage levels were similar on the bottom surfaces of slabs no. 5 and 15, although the values of midspan crack width given in Table 5.2 are significantly different (6.75 and 2.25 inches for slabs no. 5 and 15, respectively). The midspan crack widths were similar for slab no. 5 and the previously discussed slab no. 13, both of which attained midspan deflections of approximately 7.0 inches. As is evident in the photographs of Figure 5.5 and 5.13, somewhat more smoothing of the material occurred near midspan for slab no. 5 than for slab no. 13. Likewise, Figure 5.39 indicates a broad spreading of cracking on the top surface of slab no. 5. Inspection of Figure 5.15 also indicates a considerable degree of smoothing at the midspan hinge of slab no. 15.

Figures 5.23 and 5.33 indicate that, for midspan deflections of up to nearly 3.0 inches, the slab sections between the hinge lines of slabs no. 5 and 15 remained straight. These figures indicate that yielding of the principal reinforcement in slabs no. 5 and 15 occurred prior to the slabs attaining their ultimate

capacities. Also, the strain gage data indicated yielding of a stirrup near the support of slab no. 15 when the load was near the ultimate resistance of the slab. The data indicated yielding of lacing bars at locations near the support and near the midspan of slab no. 5. The yielding of the lacing in slab no. 5 occurred at load and deflection levels significantly less than that corresponding to the yielding of stirrups in slab no. 15.

Slabs No. 9 and 16

Slabs no. 9 and 16 each contained the large quantities of principal reinforcement and shear reinforcement. These slabs were pushed to support rotations of approximately 23 to 24 degrees. Figures 5.43 and 5.50 indicate that slightly more spreading of cracks occurred at the hinge lines of slab no. 9 (contained lacing) than did occur for slab no. 16 (contained stirrups), but no significant differences in damage levels were evident. As given in Table 5.3, the midspan crack width was slightly less for slab no. 9 (2.25 inches) than for slab no. 16 (crack width of 2.75 inches), although the midspan deflection was slightly greater for slab no. 9 (5.3 inches versus 5.1 inches of deflection for slab no. 16). Figure 5.53 shows that the response of the two slabs were similar throughout the load history. Slabs no. 9 and 16 did exhibit some top surface smoothing of the deformation at midspan; however, the degree of smoothing did not appear to be as great as it was for slabs no. 5 and 15.

Figures 5.27 and 5.34 present the deflection profiles for slabs no. 9 and 16, respectively. Profiles were not available for slab no. 16 for the response following the ultimate resistance

since the midspan deflection gage became disconnected during the experiment. The strain gage data indicated yielding of the principal reinforcement in both slabs prior to the slabs reaching their ultimate capacities. Figure 5.34 shows that the strain gage data indicated yielding of a stirrup near the support at a high pressure level prior to the ultimate resistance of slab no. 16; however, the strain gage (SL-1) on lacing at a location near the support of slab no. 9 did not function properly during the experiment. Figure 5.27 shows that yielding of lacing near the midspan of slab no. 9 occurred at a high pressure level prior to the ultimate resistance. Figure 5.27 indicates that the slab sections between hinge lines of slab no. 9 remained straight during the region of response shown beyond the ultimate resistance.

The large amounts of shear reinforcement in slabs no. 9 and 16 contained the concrete at large deflections (prohibited damaged concrete from falling out of the large cracks) significantly better than the small amounts of shear reinforcement in slabs no. 5 and 15. The better containment or confinement did not prohibit the rupture of the principal reinforcement; however, confinement is generally beneficial for the reduction of concrete debris that may injure personnel or damage sensitive equipment inside actual structures.

5.3 Ultimate Capacity

The method of limit analysis of reinforced concrete slabs generally known as the "yield-line theory" is usually accredited to Johansen (34). An assumed collapse mechanism consistent with

boundary conditions is used to estimate the ultimate load capacity of the slab. Johansen's yield criterion neglects the presence of any in-plane (membrane) forces in the slab. A mechanism is assumed to form when the moment capacities at critical sections have been exceeded. Segments of the slab between the critical sections (yield lines) are assumed to behave elastically with no effect on the ultimate capacity. The yield-line theory assumes that the slab has sufficient shear strength to insure a flexural collapse mode of failure.

In an often-referenced study, Ockleston (35) tested a slab in a dental hospital building and found that the interior panel of the under-reinforced floor system, acting as a restrained slab, carried more than double the load predicted by Johansen's yield-line theory. In 1958, Ockleston (36) explained that the unexpected results of his test in 1955 were not due to reinforcement strain hardening, tensile strength of concrete, nor catenary actions. He concluded that the increase in load capacity was due to the development of inplane compressive forces, termed "arching" or "dome action."

Under slowly applied uniform loading, a beam or one-way slab element initially undergoes elastic deflection. As loading continues, plastic hinges first form at the supports and later at midspan. As discussed by Park and Gamble (37), the ultimate flexural capacity is enhanced in slabs whose edges are restrained against lateral movement. This plastic theory for the load-deflection behavior of a restrained strip at and after the ultimate resistance is often referred to as "compressive-membrane theory." Full restraint against rotation and vertical translation

is assumed at the supports. Partial restraint against lateral displacement is assumed at the supports as compressive membrane action is dependent on the lateral restraint. As the slab deflects, changes in geometry cause the slab's edges to tend to move outward and to react against the stiff boundary elements. The membrane forces enhance the flexural strength of the slab sections at the yield lines. For the slabs in this study, the resistance at point "A" in Figure 5.18 corresponds to the ultimate capacity.

Two relatively difficult-to-define parameters required in the computation of the ultimate flexural resistance due to compressive-membrane action are: (a) the stiffness of the surround supporting the slab and (b) the midspan deflection occurring at ultimate capacity. Park and Gamble (37) demonstrated that the surround stiffness need not be enormous to achieve membrane action similar to that for an infinitely rigid surround. Significant membrane action occurs when the surround and the one-way slab have the same stiffness. For slabs with relatively low values of the ratio of slab length to thickness (which applies to the slabs in this study), little increase in membrane action is achieved by having a surround much stiffer than the slab. As discussed by Park and Gamble, many researchers have attempted to develop methods for determining Δ_A/t, the ratio of the deflection (Δ_A) associated with P_A to the slab thickness. This ratio is affected by the relative stiffness of the surround and slab. It is also affected by whether the slab exhibits one-way or two-way action.

Most of the research for slabs in the area of compressive-

membrane effects on ultimate capacity has concerned two-way slabs, such as in the investigations by Park (38), Morley (39), Hung and Nawy (40), Isaza (41), Brotchie, Jacobson, and Okubo (42), and Wood (43). Park (38), Morley (39), and Wood (43) assumed the central deflection at ultimate load to be 0.5 times the slab thickness for fully restrained slabs. Hung and Nawy (40) used experimental values of deflection at ultimate load and noted that the ultimate load was not always reached at a deflection equal to 0.5 times the slab thickness. Instead, values ranging from approximately 0.4 to 1.0 times the slab thickness were considered. Work by Isaza (41) indicated that the ultimate capacity, enhanced by compressive membrane action, occurred at a midspan deflection of approximately one-sixth of the slab thickness. The range given for the Δ_A/t ratio is broad when these researchers' values, which varied from 0.17 to 1.0, are considered.

In addition to the previous investigations conducted at WES and NCEL as described in Chapter 3 of this thesis, only a few investigators have studied the behavior of one-way slabs. Roberts (44) and Christiansen (45) loaded longitudinally restrained, conventionally reinforced one-way slabs. The ultimate capacity varied from 1.5 to 17 times the Johansen yield-line load. The enhancement beyond the yield-line load increased as the concrete strength was increased. It also increased as the principal reinforcement ratio decreased.

A computer program, consistent with the theory as presented by Park and Gamble, was written during this study to compute the enhancement due to compressive-membrane action for the slabs. The theory is based on the equilibrium and deformations of a slab

strip as shown in Figure 5.54. Park and Gamble showed that the sum of internal moments, including thrusts, can be expressed as follows:

$$M_u' + M_u - n_u\delta = 0.85f_c'\beta_1 h \left[\frac{h}{2}\left(1 - \frac{\beta_1}{2}\right) + \frac{\delta}{4}(\beta_1 - 3)\right.$$

$$+ \frac{\beta L^2}{4\delta}(\beta_1 - 1)\left(\epsilon + \frac{2t}{L}\right) + \frac{\delta^2}{8h}\left(2 - \frac{\beta_1}{2}\right) + \frac{\beta L^2}{4h}\left(1 - \frac{\beta_1}{2}\right)\left(\epsilon + \frac{2t}{L}\right)$$

$$\left. - \frac{\beta_1\beta^2 L^4}{16h\delta^2}\left(\epsilon + \frac{2t}{L}\right)^2\right] - \frac{1}{3.4f_c'}(T' - T + C_s' + C_s)^2$$

$$+ (C_s' + C_s)\left(\frac{h}{2} - d' - \frac{\delta}{2}\right) + (T' + T)\left(d - \frac{h}{2} + \frac{\delta}{2}\right)$$

In the above expression, ϵ represents the sum of elastic, creep, and shrinkage strains and t is the lateral movement of one support. M_u and M_u' are the ultimate moments of resistance along the plastic hinge lines at midspan and the supports, respectively. All other terms are represented in Figure 5.54. For a pure three-hinge mechanism, β has a value of 0.5 and the displacement, δ, is simply the midspan deflection that has be termed Δ in this thesis.

When external forces are included and the principle of virtual work is applied for a slab of width B, the following equation results to express the resistance of the slab to a uniform load, w.

$$w = 8\ (M_u' + M_u - n_u\Delta)/L^2B$$

The ultimate capacities experimentally obtained for the sixteen slabs are summarized in Table 5.6. In Table 5.6, the slabs are grouped according to their reinforcement details (first

by principal reinforcement quantity, and second by shear reinforcement quantity). Similarly, Figures 5.55 through 5.57 are composite presentations of smoothed (for visual clarity) midspan load-deflection curves for the slabs, grouped by reinforcement details. Since the complete midspan load-deflection curves were not available for slabs no. 12, 14, and 16, the reader will also be referred back to Figures 5.51 through 5.53 in the following discussion for a comparison of data recorded at the quarterspan location. As noted in Table 5.6, the region of the data near the ultimate capacity for slab no. 1 was not recorded due to a low range setting for the data-acquisition software (slab no. 1 was the first slab tested in this series). It is obvious from the shape of the midspan load-deflection curve of slab no. 1 (presented in Figure 5.55) that the maximum recorded resistance of 57 psi was near the slabs's ultimate capacity and that the actual ultimate capacity was probably in the range of 60 to 65 psi. The computed Johansen yield-line values are given in Table 5.6 for comparison to the experimentally-obtained ultimate capacities. It appears that compressive membrane forces acted to increase the ultimate capacities of the slabs from approximately 1.2 to 4.0 times the computed yield-line strengths. Also, the Δ_A/t ratio varied among the slabs with values from approximately 0.15 to 0.37. The lowest values obtained for Δ_A/t corresponded to slabs no. 1 and 3. Since slab no. 3 failed in shear prior to attaining its potential ultimate flexural capacity, the inclusion of its Δ_A/t value of 0.15 in the calculation of an average value is not appropriate. Also, it appears that slab no. 1 experienced a double peak in the region of ultimate capacity. The midpoint of

its double-peak region corresponds to a Δ_A/t value of approximately 0.25. With the elimination of slabs no. 1 and 3, the average of the Δ_A/t values presented in Table 5.6 for the remaining fourteen slabs is approximately 0.29. There was no consistent pattern for the values of Δ_A/t in relation to the slab construction parameters. However, it is apparent from the P_A/YL ratio given in Table 5.6 that the compressive membrane enhancement was greatest for the slabs with the smallest principal reinforcement ratio. The compressive membrane enhancement was slightly greater for the group of slabs with the medium quantity of principal steel than for that with the large quantity.

From an inspection of Table 5.6, as well as Figures 5.51 through 5.53 and Figures 5.55 through 5.57, it appears that the ultimate capacities of the slabs were affected by shear reinforcement characteristics as exhibited by the following pattern: the baseline slabs (nos. 1, 2, and 3) had the lowest values, followed in increasing order by the corresponding slab with stirrups, and then by the corresponding slab with lacing. For example, the ultimate capacities of slabs no. 1, 10, and 4 were 57 (actually near 60 to 65), 63, and 71 psi, respectively. This pattern was also demonstrated by slabs no. 3 (failed in shear prior to reaching the potential ultimate flexural capacity), 15, and 5; slabs no. 1, 11, and 6; and slabs no. 3, 16, and 9. However, this pattern did not hold for slabs no. 2, 12, 13, and 7, all of which contained medium amounts of both principal and shear reinforcement (except for no shear reinforcement in baseline slab no. 2). Slabs no. 2, 12, 13, and 7 displayed ultimate capacities of 87, 85, 89, and 83 psi, respectively. Additionally, the

ultimate capacities were approximately equal for slabs no. 8 (contained lacing) and 14 (contained stirrups), both of which contained a small amount of principal reinforcement and a large amount of shear reinforcement.

The results of the application of compressive-membrane theory to the slabs in this study are presented in Table 5.7. The experimental values given in Table 5.6 for the ultimate capacity (P_A) and the Δ_A/t ratio are repeated in Table 5.7 for ease of comparison. For each slab, the table presents the ultimate capacities computed using compressive membrane theory with three different assumed values of Δ_A/t: (a) the experimental value taken from Table 5.6, (b) a relatively low value of 0.1, and (c) a relatively high value of 0.5. Actually, only Δ_A is substituted into the theory. The Δ_A/t ratio is used in this discussion for consistency with the nomenclature typically used when compressive membrane theory is discussed in the literature. The Δ_A/t ratio is convenient as a parameter for comparing data and for the designer that wants to consider compressive membrane behavior by relating existing data to his slab of some thickness. Figures 5.58 through 5.73 present the computed ultimate capacities, corresponding to more assumed values of Δ_A/t than given in Table 5.7, plotted with the experimentally obtained midspan load-deflection curves. In particular, these figures visually indicate how well the compressive membrane theory predicts the ultimate capacities of the slabs when the experimentally-obtained Δ_A/t values are used. Figures 5.58 through 5.73 each show a horizontal line plotted at the computed yield-line resistance values given in Table 5.6. The sloping lines, representing tensile-membrane behavior, in these

figures will be discussed later.

The values given in Table 5.7 show that the compressive membrane theory closely predicts the ultimate capacities of most of the slabs having ρ values of 0.0025 and 0.0056 (the first two groups of slabs in the table) when the experimental values for Δ_A/t are used. Of these two groups, the slab with the greatest discrepancy between experimental and computed values is slab no. 6. The experimentally obtained ultimate capacity of slab no. 6 was significantly greater than the other slabs that had a ρ of 0.0025. As shown in Table 5.7, compressive-membrane theory closely predicts the ultimate capacity of slab no. 6 when a Δ_A/t ratio of 0.1 is used.

For the slabs with a ρ of 0.0097, compressive-membrane theory more closely predicts the experimentally obtained ultimate capacities when a Δ_A/t value of 0.1 is used rather than the experimental values of Δ_A/t. As shown by the experimental data curve in Figure 5.60, an abrupt drop in resistance (shear failure) occurred for slab no. 3 prior to the attainment of the potential flexural capacity. The failure occurred at a load resistance of approximately 106 psi, which is approximately 74 to 83 percent of the ultimate capacities exhibited by the other slabs that had a ρ value of 0.0097.

5.4 Reserve Capacity

As discussed by Park and Gamble (37), after the ultimate load resistance has been reached in a reinforced concrete slab, the load resistance decreases until membrane forces in the central region of the slab change from compression to tension. In pure

tensile membrane behavior, cracks penetrate the whole thickness, and yielding of the steel spreads throughout the central region of the slab. The load is carried mainly by reinforcing bars acting as a tensile net or membrane. For a one-way slab, the reinforcement must be sufficiently anchored or restrained at the supports to allow development of the membrane forces. This action typically results in an increase in load resistance, often called "reserve capacity," at large deflections. Reserve capacity is important in the design of protective structures since moderate to severe damage is often acceptable if collapse is avoided. It is possible for a slab's peak reserve capacity to equal or be greater than the ultimate capacity. For each slab in this study, the reserve capacity was less than the ultimate capacity.

The P_D values given in Table 5.2 represent the peak reserve capacities attained by each slab. Maximum deflections (ideally corresponding to the peak reserve capacities) measured posttest at midspan, as presented in Table 5.1, differ from the deflection values given in Table 5.2. Values presented in Table 5.1 were manually measured after each experiment while those in Table 5.2 were electronically recorded during the experiments. As discussed in Section 5.2, a comparison of the electronically recorded maximum deflections with the posttest measured deflections is presented in Table 5.4.

Park and Gamble (37) presented the tensile-membrane theory using standard membrane theory with the following assumptions: (a) all concrete has cracked throughout its depth and is incapable of carrying any load, (b) all of reinforcement has reached the yield strength and acts as a plastic membrane, (c) no strain

hardening of steel occurs, and (d) only the reinforcement that extends over the whole area of the slab contributes to the membrane. The theory assumes that tensile-membrane action is dependent on yield forces in the steel. It does not account for combined bending and tensile-membrane action. For a one-way slab, the theory results in the following relationship:

$$\Delta = wL^2/8T_y$$

where Δ = midspan deflection

w = uniform load per unit area

T_y = yield force of the reinforcement per unit width

L = clear span length

Figures 5.58 through 5.73 include plotted lines representing the theoretical tensile-membrane slope for each slab. Each of Figures 5.58 through 5.73 includes a plotted single point representing the peak reserve capacity (same as P_D in Table 5.2) and the posttest measured maximum deflection taken from Table 5.4. Two cases are presented in these figures for predicted tensile-membrane slopes: (a) all principal reinforcement in each face is assumed to contribute to tensile-membrane action, and (b) only one-half of the principal reinforcement is assumed to contribute to tensile-membrane action. Case (b) might represent the condition where all principal reinforcement in the bottom face at midspan has ruptured while all principal reinforcement in the top face at the supports has ruptured. This condition requires that the integrity of the slab sections between hinge lines is well-maintained and significant pull-out or slip of the ruptured

reinforcement is avoided. Except for slabs no. 2 and 3, the slabs in this study are represented well by case (b) above as a 3-hinge mechanism was formed and nearly all reinforcement in the tension faces at the regions of the yield lines ruptured.

For each of the slabs with the smallest principal reinforcement ratio (ρ = 0.0025), the predicted tensile-membrane slope that accounts for one-half of the principal reinforcement is near to the experimental curve at the low point (point C in Figure 5.18) that occurs prior to the increase in resistance. The best match of the tensile-membrane slope being tangent to the experimental curve occurs for slab no. 1. As shown in Figure 5.55, the dips in the load-deflection curves were lowest (in terms of load resistance) for slabs no. 10, 11, and 14, which were the only slabs containing stirrups in this group. Figure 5.51 shows that the shapes of the quarterspan load-deflection curves for slabs no. 8 (contained lacing) and 14 (contained stirrups) were very similar; but, slab no. 8 maintained a resistance approximately 3 psi greater than did slab no. 14. All of the slabs with a ρ value of 0.0025 experienced an increase in resistance up to or slightly higher than the Johansen yield-line resistance. Also, the predicted tensile-membrane slope accounting for all of the principal reinforcement intersects (or nearly intersects) the experimental curve at the point of peak reserve capacity (point D in Figure 5.18) for each of these slabs. This intersection at the peak reserve capacity is merely a coincidence since the predicted tensile-membrane slope that accounts for all of the reinforcement is exactly twice that of the slope for when only half of the principal reinforcement is considered. If

strain-hardening of the principal reinforcement is taken into account in accordance with the material property data presented in Table 4.4, the predicted tensile-membrane slopes may be increased by approximately 8 percent. An eight percent increase in the slope associated with one-half of the principal reinforcement is much less than the approximately 100 percent increase required to match the peak reserve capacities of the slabs that had a ρ value of 0.0025.

As previously mentioned, the load resistance of the slabs with stirrups within the group of slabs that had a ρ value of 0.0025 (Figure 5.55) dropped to lower values after reaching the ultimate capacity than the companion slabs with lacing or no shear reinforcement. Similarly, the peak reserve capacity values for the slabs shown in Figure 5.55 with stirrups were less than the values for the slabs with lacing. The peak reserve capacity for slab no. 1 (no shear reinforcement) was similar to that of the slabs with stirrups. These values are presented in Table 5.8 for ease of comparison and indicate that the lacing bars were more effective than the stirrups in enhancing tensile-membrane behavior. It is plausible that the lacing bars contributed to the reserve capacity due to their continuity throughout the length of the slab. The lacing bars bridged across the dominate cracks at midspan. The tensile strength of the lacing was apparently not fully mobilized during the tensile-membrane region of response (none of the lacing bars ruptured). In fact, strain gage data indicated that if shear reinforcement did not yield prior to a slab reaching its ultimate capacity, then it usually did not yield later during the experiment. Exceptions to this observation are

lacing in slab no. 5 (based on gages SL-3 and SL-5) and stirrups in slabs no. 12 (SS-1) and 13 (SS-4).

The composite of the midspan load-deflection curves for the small group of four slabs with the medium amount ($\rho = 0.0056$) of principal reinforcement is presented in Figure 5.56. The quantity of shear reinforcement was not varied among these slabs except that slab no. 2 did not contain any shear reinforcement. The values in Table 5.9 indicate that the slabs containing shear reinforcement achieved similar values of peak reserve capacity. Figure 5.56 shows that each of the three slabs of this group that contained shear reinforcement maintained a resistance of 20 to 25 psi during the transition region immediately prior to tensile-membrane action. Since the midspan deflection gage connection deteriorated during the experiment, Figure 5.52 is used to compare the response in the tensile-membrane region for slabs no. 7 and 12 using data recorded at the one-quarterspan. Due to the inaccuracies of the deflection gage measurements at large deflections, the composite figures do not provide a complete picture for the region of reserve capacity. The posttest measured deflections must be considered as shown in Figures 5.64, 5.69, and 5.70 for slabs no. 7, 12, and 13, respectively. Figures 5.64 and 5.69 indicate that, as in the case of the slabs having a ρ value of 0.0025, the two slopes given for the tensile membrane region bracket (bound) the single point that represents the posttest measured deflection and peak reserve capacity. Unlike for the slabs with a ρ value of 0.0025, the peak reserve capacities of slabs no. 7 and 12 are nearer to the predicted tensile-membrane slope that accounts for one-half of the principle steel. The peak

reserve capacity for slab no. 13 (Figure 5.70) plots a little below this tensile membrane slope. Also unlike the slabs with a ρ value of 0.0025, the reserve capacities for these three slabs (nos. 7, 12, and 13) did not reach the Johansen yield-line value. The lacing bars did not appear to affect the reserve capacity of the slabs with a ρ value of 0.0056 to a different degree than did stirrups.

Figure 5.57 presents the composite of the midspan load-deflection curves for the slabs with the large amount (ρ = 0.0097) of principal reinforcement. The peak reserve capacity values for the slabs shown in Figure 5.57 are presented in Table 5.10 for comparison. It is apparent from Figure 5.57 that slab no. 5 behaved slightly different from the other slabs with shear reinforcement. The curve for slab no. 5 flattened at a resistance of approximately 68 psi after reaching ultimate capacity. After approximately one inch of additional deflection, the resistance gradually decreased and only increased slightly in the tensile-membrane region. Figure 5.53 indicates that responses of slab no. 9 (contained lacing) and slab no. 16 (contained stirrups) were very similar. Slab no. 16 achieved a slightly greater peak reserve capacity. In fact, an inspection of Table 5.10 indicates that, for all slabs having a ρ value of 0.0097, the slabs containing stirrups achieved a greater value of peak reserve capacity than did the laced slabs.

Figures 5.62, 5.66, 5.72, and 5.73 indicate that the tensile-membrane slope that accounts for one-half of the principal steel very closely approaches the peak reserve capacities (based on the posttest measured deflections) of slabs no. 9, 15, and 16, but not

slab no. 5. The best fits occur for the two slabs with stirrups (nos. 15 and 16).

It appears from this discussion, and is evident from the values given in Tables 5.8, 5.9, and 5.10, that peak reserve capacity was best enhanced by lacing in the slabs with a ρ value of 0.0025, but by stirrups in the slabs with a ρ value of 0.0097. The two types of shear reinforcement appeared to be equally effective in slabs having the medium ρ value of 0.0056.

5.5 Response Limits

Although the sixteen slabs discussed in this paper were statically loaded, their failure modes are assumed to be similar to what would occur from dynamic test conditions (except for close-in detonations). In particular, similar failure modes should be expected in slabs located at distances far enough from the explosive source such that loading occurs primarily from the quasi-static loading (gas pressure) that accompanies internal detonations.

All slabs (except for slabs no. 2 and 3 which did not contain shear reinforcement and experienced shear failures) sustained support rotations greater than 20 degrees. A support rotation of 20 degrees corresponds to "heavy" damage by the criteria given in ETL 1110-9-7. The ETL criteria are for when the scaled range is greater than 0.5 ft/lb$^{1/3}$, thereby eliminating the cases of very close-in and contact detonations. Additionally, the criteria are for slabs with L/d ratios greater than 5 and principal steel spacings not greater than d. Also, stirrups are required at a spacing not greater than d/2 when the scaled range is less than

2.0 ft/lb$^{1/3}$ and at a spacing less than d at larger scaled ranges.

Only slabs no. 8, 9, 14, and 16 had shear reinforcement spaced at d/2. The stirrups or lacing in the other slabs that had shear reinforcement were spaced at d or 3d/4. Although slab no. 1 had no shear reinforcement, it sustained support rotations that exceeded the response limits given in ETL 1110-9-7 without complete collapse. Slabs no. 1, 2, and 3 (no shear reinforcement) did not meet the construction criteria given in the ETL since it requires that a minimum of 50 psi shear stress capacity be provided by shear reinforcement. All of the slabs contained principal reinforcement spaced at less than d as required by the ETL.

The experimental results support the "heavy damage" response limits given in the ETL. In particular, the experiments indicate that lacing is not required for slabs to be capable of achieving the allowable response limits of the ETL, considering the mentioned restrictions for use of the ETL criteria. By restricting the use of the ETL to slabs that are loaded at scaled distances greater than 0.5 ft/lb$^{1/3}$ and have L/d ratios greater than 5, an attempt is made to avoid failure modes that are dominated by shear. The fact that not all of the slabs satisfied each of the construction criteria parameters of the ETL, but did sustain support rotations greater than 20 degrees, indicates some conservatism in the ETL criteria. Some conservatism is desirable for criteria in design documents, but these experiments indicate that the allowable response limits of TM 5-1300 are overly conservative. TM 5-1300 allows support rotations of only 4 degrees (8 degrees if tensile membrane forces can be developed)

for slabs with stirrups and 12 degrees for slabs with lacing. It is recognized that, while ETL 1110-9-7 is intended for use in the design of military facilities that may be subjected to conventional weapons effects, TM 5-1300 has a broad application. Its use ranges from the design of military facilities for the storage of weapons to the design of civilian facilities used in explosives manufacturing. Therefore, a sweeping change in TM 5-1300 response limits is not practical. Instead, the TM 5-1300 design criteria should be more varied, depending on the application. Less conservative response limits should be allowed for elements of facilities where large deflections are not detrimental to the purpose of the structure. An example may be the design of an explosives storage facility where the propagation of an accidental explosion from storage bay to storage bay is unacceptable, and structural collapse of divider walls must be avoided. Particularly related to this study, there is enough previous dynamic test data, when combined with the results of the experiments performed in this investigation, to indicate that lacing is not necessary for the allowance of large deflections (corresponding to support rotations of 12 to 20 degrees) in at least some applications of TM 5-1300. Realistically, a thorough series of dynamic experiments are needed for extending this study to the development of accurate design criteria for all applications of TM 5-1300.

Table 5.1. Support Rotation and Ratio of Midspan Deflection to Clear Span (Based on Posttest Measurements)

Slab	Midspan Deflection (Δ) (inches)	Δ/L (percent)	θ (degrees)
1	4.4*	18.3	20.1
4	5.5	22.9	24.6
10	5.0	20.8	22.6
6	5.5	22.9	24.6
11	5.9	24.6	26.2
8	5.5	22.9	24.6
14	5.7	23.8	25.4
2**	1.5	7.1	8.1
7	4.5	18.8	20.6
12	5.7	23.8	25.4
13	7.0	29.2	30.3
3***	2.2	9.2	10.4
5	7.0	29.2	30.3
15	5.3	22.1	23.8
9	5.3	22.1	23.8
16	5.1	21.3	23.0

* Presented deflection values were manually measured following removal of the neoprene membrane.

** Experiment terminated early due to water leak.

*** Slab failed in shear.

Table 5.2. Midspan Load-Deflection Summary

Slab	P_A (psi)	Δ_A (in)	P_B (psi)	Δ_B (in)	P_C (psi)	Δ_C (in)	P_D (psi)	Δ_D (in)
1	57*	0.52	8	2.41	8	2.41	23	3.61****
4	71	0.80	10	2.31	10	2.96	31	4.36
10	63	0.65	3	2.33	8	3.59	25	4.77
6	88	0.79	10	2.58	10	2.58	31	4.80
11	63	0.91	2	2.65	2	2.65	22	5.00
8	64	1.00	8	2.50	8	3.10	26	4.50
14	64	0.87	4	2.60	**	**	23***	**
2	87	0.80	44	1.10	44	1.10	53	1.65
7	83	0.88	38	2.32	11	3.61	43	4.00
12	85	1.10	19	3.10	**	**	43***	**
13	89	0.74	25	2.00	25	3.19	41	4.63
3	106	0.45	59	0.51	59	0.51	88	2.18
5	135	0.89	70	1.69	27	3.88	41	4.96
15	130	0.81	58	2.30	14	3.11	75	4.00
9	137	0.91	17	2.85	17	2.85	73	4.22
16	**	**	**	**	**	**	79***	**

* Actual experimental value was greater than shown due to data record clip during experiment.

** Large crack formed directly at deflection gage connection on slab, causing loss of connection.

*** Taken from data recorded at the one-quarterspan location.

**** Deflection values presented were electronically recorded during the experiment.

Table 5.3. Crack Widths

Slab	Top Crack Left Support (inches)	Bottom Crack Midspan (inches)	Top Crack or Crushed Area, Midspan (inches)	Top Crack Right Support (inches)
1	1.38*	1.88	1.5	1.13
2	0.25	NA	NA	NA
3	4.0	NA	NA	NA
4	1.13	2.5	2.5	1.13
5	3.0	6.75	5.0	3.0
6	1.25	3.5	2.5	1.5
7	0.88	2.13	2.5	0.88
8	1.5	3.13	2.5	1.25
9	3.0	2.25	2.0	1.25
10	1.0	2.13	2.5	1.25
11	1.25	3.38	3.25	1.5
12	2.0	4.0	4.5	1.75
13	3.0	6.75	5.0	3.0
14	1.25	3.25	2.0	1.25
15	1.25	2.25	5.5	2.0
16	1.25	2.75	2.0	1.25

* All crack widths were measured on slab surface following removal of neoprene membrane.

NOTE: The left support is taken to be that on one's left hand side when looking at the slab from the side with the reaction structure's removable door (the view shown in Figure 5.1)

Table 5.4. Maximum Midspan Deflection

Slab	Posttest Measured* Midspan Deflection (M) (in)	Electronically* Recorded Midspan Deflection (R) (in)	R/M
1	4.4	3.61	0.82
4	5.5	4.36	0.79
10	5.0	4.77	0.95
6	5.5	4.80	0.87
11	5.9	5.00	0.85
8	5.5	4.50	0.82
14	5.7	-	-
2	1.7	1.65	0.97
7	4.5	4.00	0.89
12	5.7	-	-
13	7.0	4.63	0.66
3	2.2	2.18	0.99
5	7.0	4.96	0.71
15	5.3	4.00	0.75
9	5.3	4.22	0.80
16	5.1	-	-

* Deflection value manually measured after completion of each experiment.

** Deflection value taken from data plots recorded during each experiment.

Table 5.5. Quarter-span Load-Deflection Summary

Slab	P_A (psi)	Δ_A (in)	P_B (psi)	Δ_B (in)	P_C (psi)	Δ_C (in)	P_D (psi)	Δ_D (in)
7	83	0.43	22	1.05	11	1.70	43	1.98
12	85	0.56	25	1.13	19	1.85	43	2.51
8	64	0.45	8	1.10	9	1.43	26	2.13
14	64	0.45	4	1.14	4	1.54	23	2.14
9	137	0.48	17	1.36	17	1.36	73	2.28
16	132	0.51	7	1.32	7	1.32	79	2.37

Table 5.6. Measured Ultimate Capacity

Slab	Yield Line YL	P_A (psi)	P_A/YL (psi)	Δ_A (in)	Δ_A/t	Shear Rein. (ρ_{shear})	$\rho_{tension}$
1	22	57*	2.6	0.57	0.19	none	0.0025
4	22	71	3.2	0.80	0.27	lacing (0.0026)	0.0025
10	22	63	2.9	0.65	0.22	stirrups (0.0026)	0.0025
6	22	88	4.0	0.79	0.26	lacing (0.0034)	0.0025
11	22	63	2.9	0.91	0.30	stirrups (0.0034)	0.0025
8	22	64	2.9	1.00	0.33	lacing (0.0052)	0.0025
14	22	64	2.9	0.87	0.29	stirrups (0.0052)	0.0025
2	53	87	1.6	0.80	0.27	none	0.0056
7	53	83	1.6	0.88	0.29	lacing (0.0036)	0.0056
12	53	85	1.6	1.10	0.37	stirrups (0.0036)	0.0056
13	53	89	1.7	0.74	0.25	stirrups (0.0036)	0.0056
3**	92	106	1.2	0.45	0.15	none	0.0097
5	92	135	1.5	0.89	0.30	lacing (0.0031)	0.0097
15	92	130	1.4	0.81	0.27	stirrups (0.0031)	0.0097
9	92	137	1.5	0.91	0.31	lacing (0.0063)	0.0097
16	92	132	1.4	-	-	stirrups (0.0063)	0.0097

* Actual experimental value was greater than shown due to data record clip during experiment.

** Shear failure occurred prior to attainment of potential flexural capacity.

Table 5.7 Compressive Membrane Analyses

Slab	Experimental Ultimate Resistance (P_A) (psi)	Experimental Δ_A/t	Predicted (Computed) Ultimate Resistance (P_A) Using Compressive Membrane Theory: For Δ_A/t= Experimental Δ_A/t (psi)	For Δ_A/t= 0.1 (psi)	For Δ_A/t= 0.5 (psi)
1	57*	0.19	77	89	40
4	71	0.27	67	89	40
10	63	0.22	73	89	40
6	88	0.26	69	89	40
11	63	0.30	63	89	40
8	64	0.33	59	89	40
14	64	0.29	64	89	40
2	87	0.27	86	105	63
7	83	0.29	83	105	63
12	85	0.37	75	105	63
13	89	0.25	88	105	63
3**	106	0.15	123	128	95
5	135	0.30	109	128	95
15	130	0.27	112	128	95
9	137	0.31	108	128	95
16	132***	0.31****	108	128	95

* Actual experimental value was greater than shown due to data record clip during experiment.

** Shear failure occurred prior to attainment of potential ultimate flexural capacity.

*** Based on one-quarter span load-deflection curve.

**** Based on mid-span deflection measured for similar slab (slab no. 9).

Table 5.8 Peak Reserve Capacity for Slabs with ρ of 0.0025

Slab	Shear Reinforcement ρ_{shear}	P_D (psi)
1	none	23
4	lacing 0.0026	31
10	stirrups 0.0026	25
6	lacing 0.0034	31
11	stirrups 0.0034	22
8	lacing 0.0052	26
14	stirrups 0.0052	23

Table 5.9. Peak Reserve Capacity for Slabs with ρ of 0.0056

Slab	Shear Reinforcement ρ_{shear}	P_D (psi)
2	none	53*
7	lacing 0.0036	43
12	stirrups 0.0036	43**
13	stirrups 0.0036	41

* Reflects an increase in resistance following shear failure. Experiment was terminated early due to water leak.

** Taken from one-quarter span data.

Table 5.10. Peak Reserve Capacity for Slabs with ρ of 0.0097

Slab	Shear Reinforcement ρ_{shear}	P_D (psi)
3	none	88*
5	lacing 0.0031	41
15	stirrups 0.0031	75
9	lacing 0.0063	73
16	stirrups 0.0063	79**

* Reflects an increase in resistance following shear failure.

** Taken from one-quarter span data.

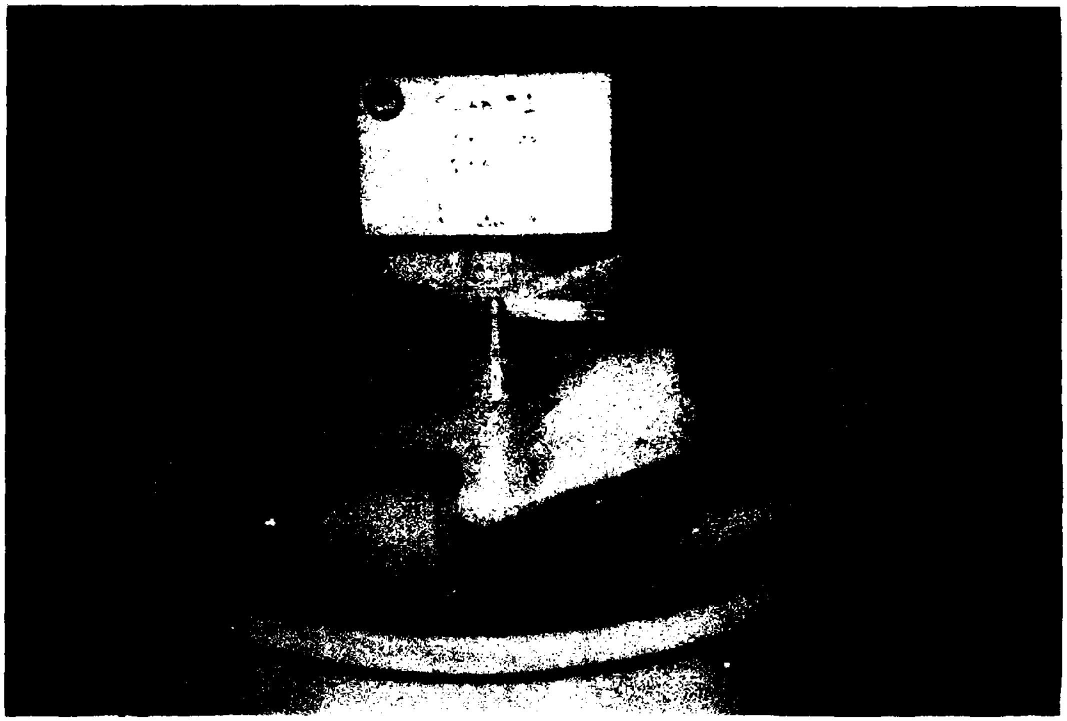


Figure 5.1. Posttest View of Slab No. 1

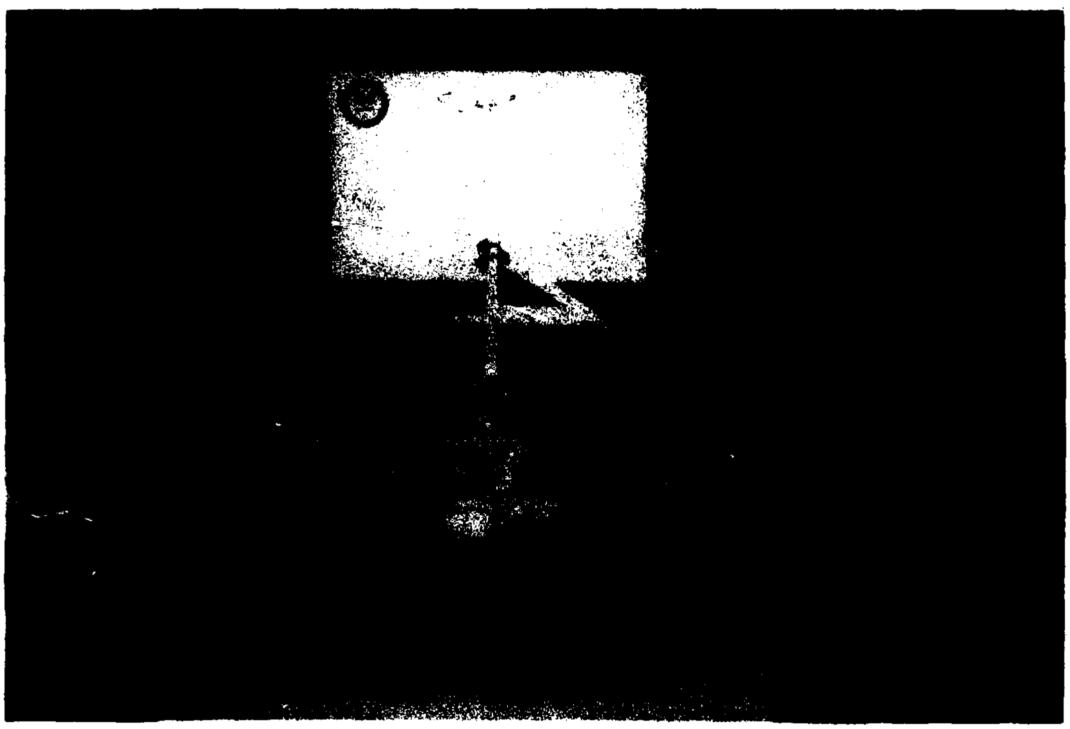


Figure 5.2. Posttest View of Slab No. 2

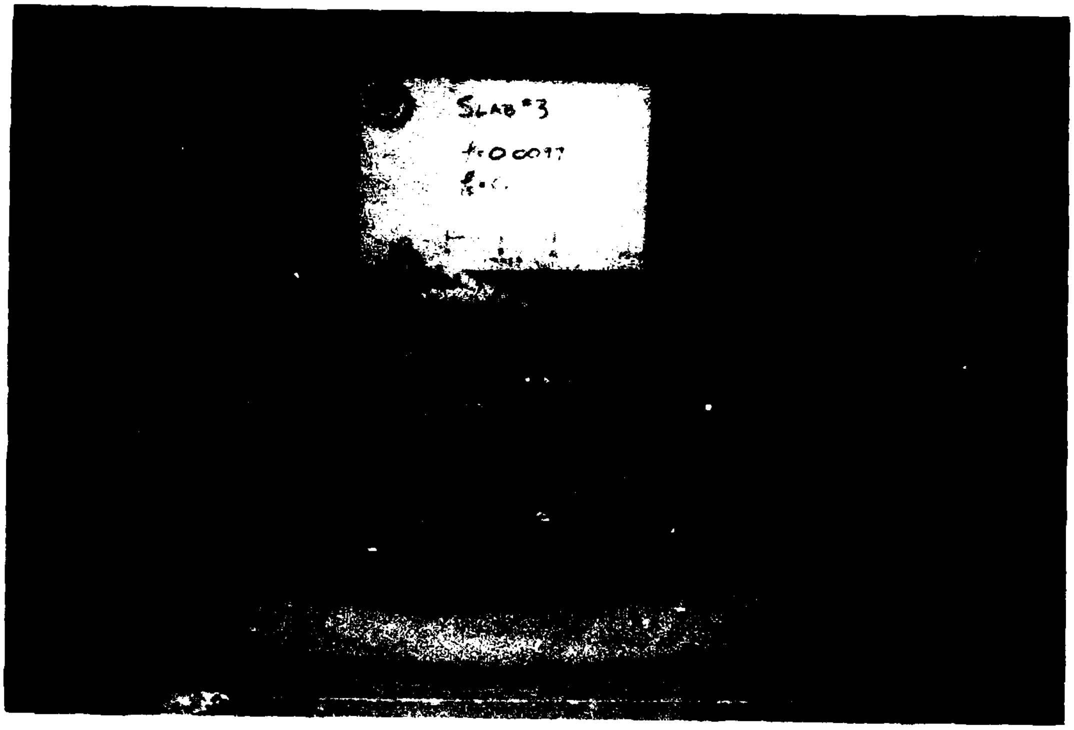


Figure 5.3. Posttest View of Slab No. 3

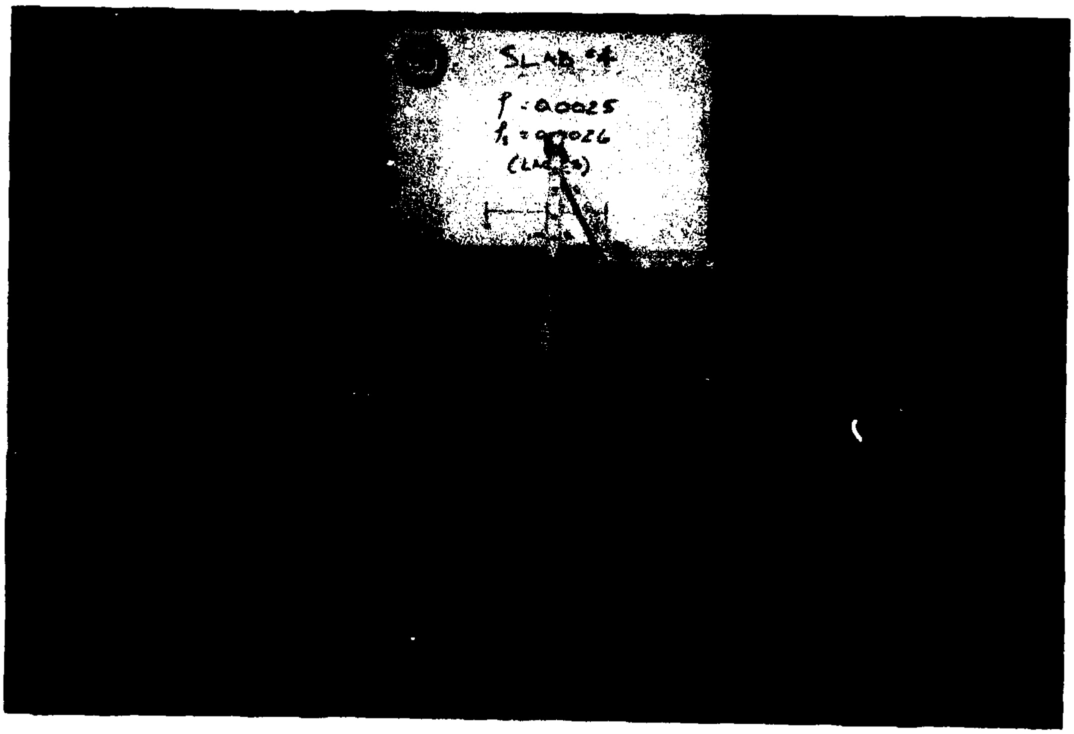


Figure 5.4. Posttest View of Slab No. 4

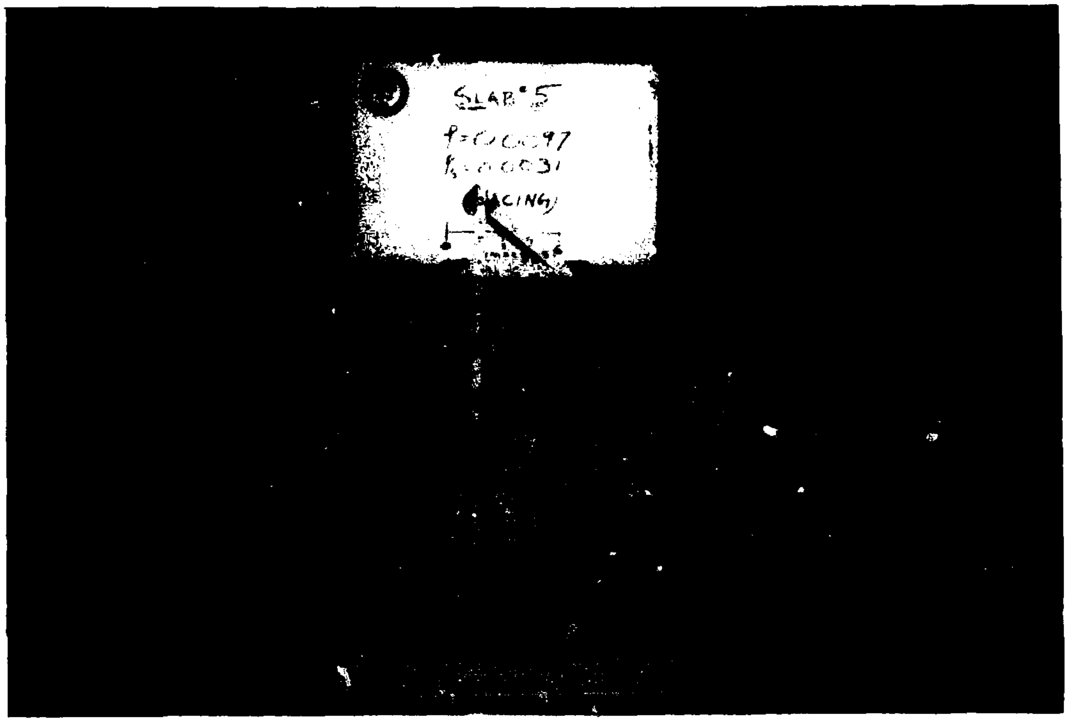


Figure 5.5. Posttest View of Slab No. 5

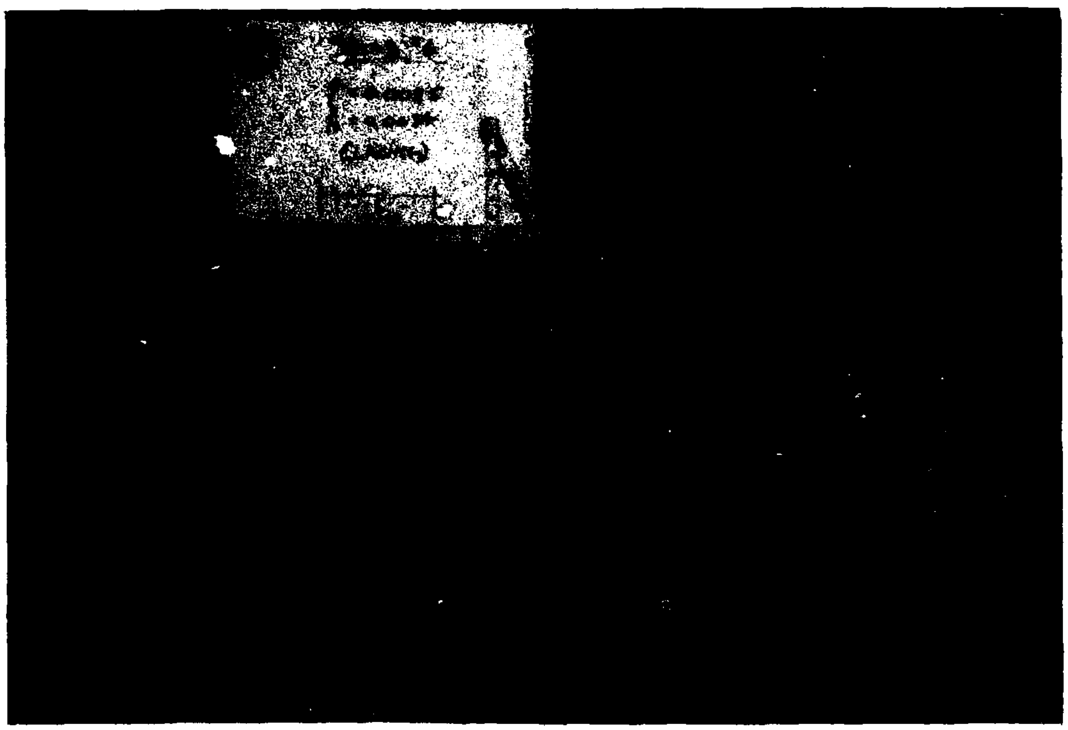


Figure 5.6. Posttest View of Slab No. 6

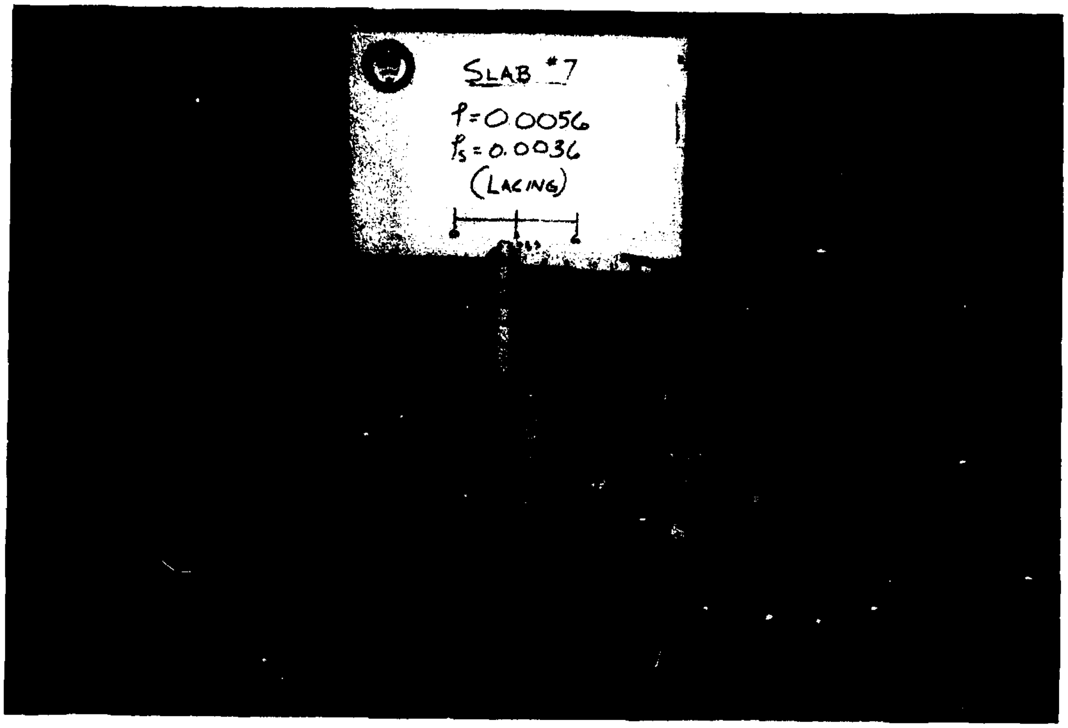


Figure 5.7. Posttest View of Slab No. 7

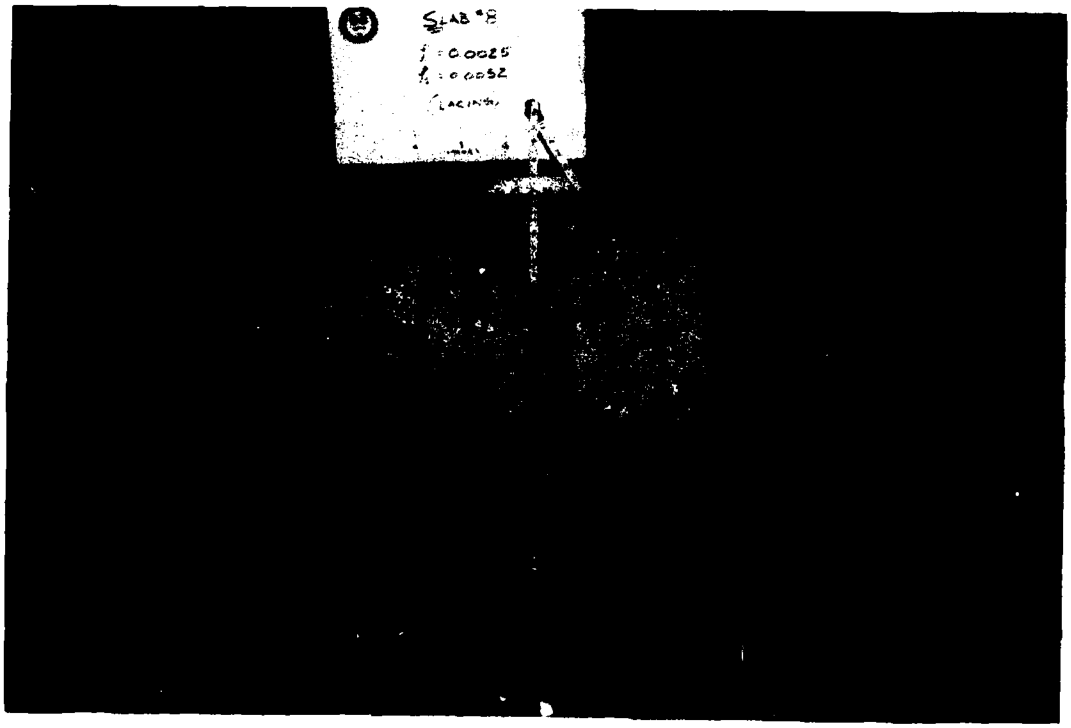


Figure 5.8. Posttest View of Slab No. 8

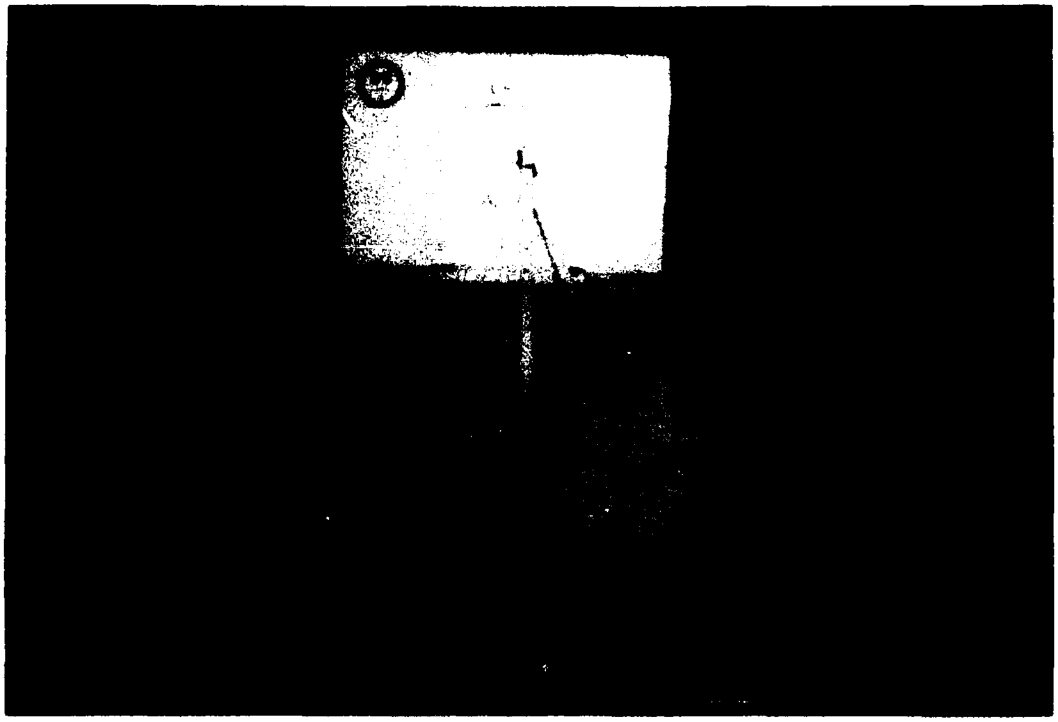


Figure 5.9 Posttest View of Slab No. 9

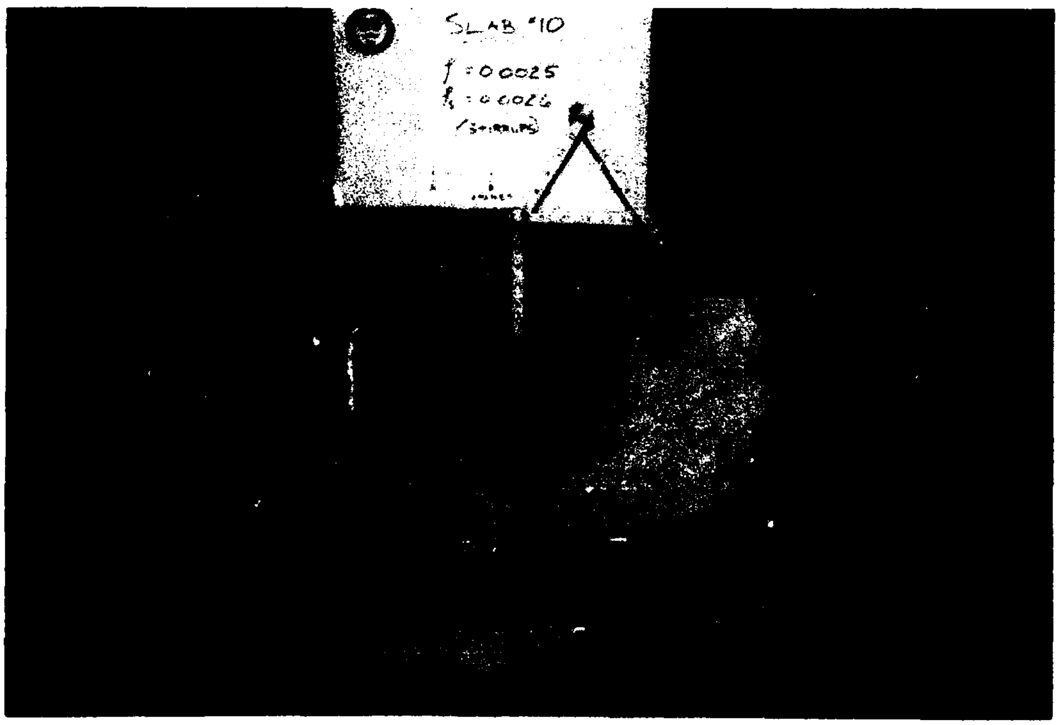


Figure 5.10. Posttest View of Slab No. 10

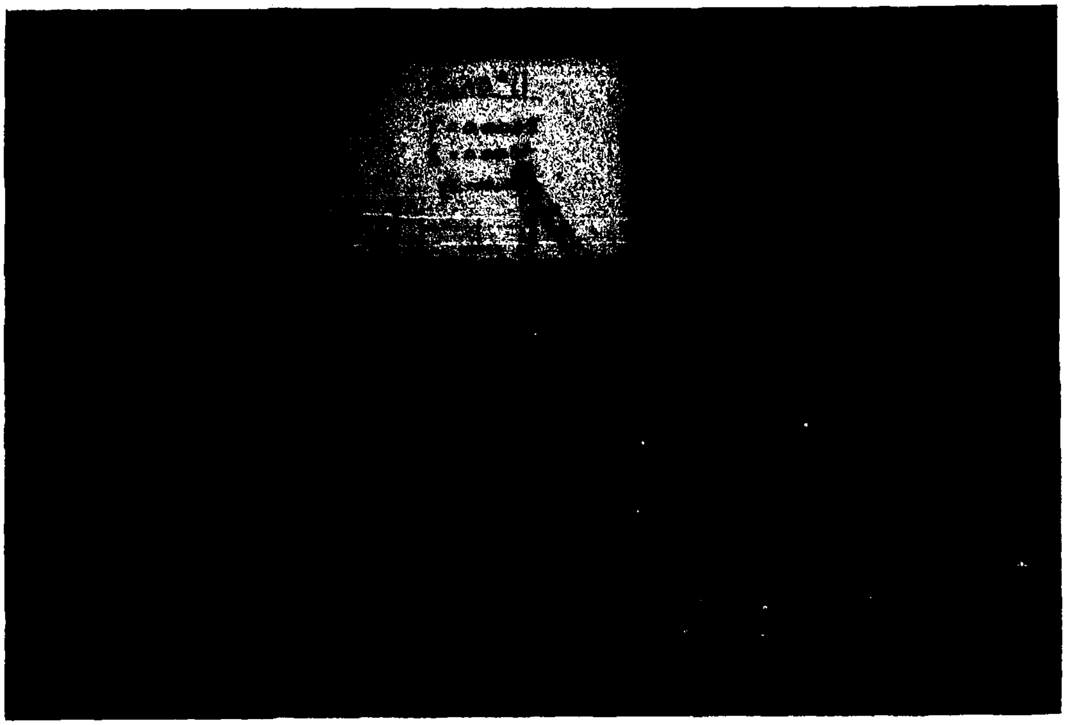


Figure 5.11. Posttest View of Slab No. 11

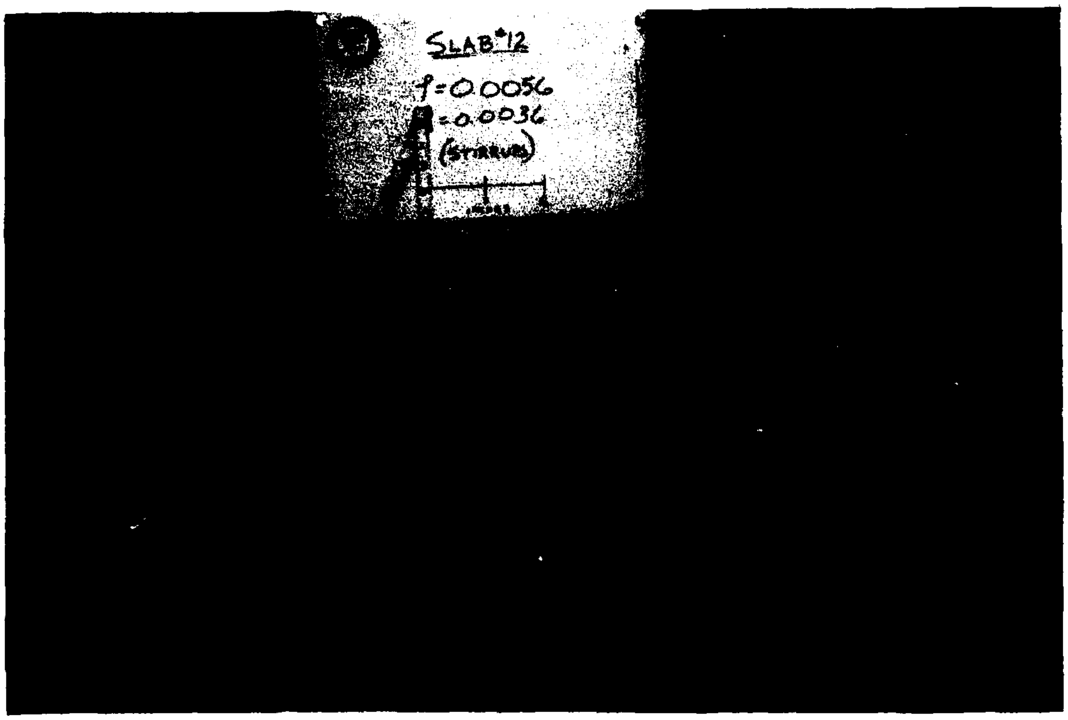


Figure 5.12. Posttest View of Slab No. 12

Figure 5.13. Posttest View of Slab No. 13

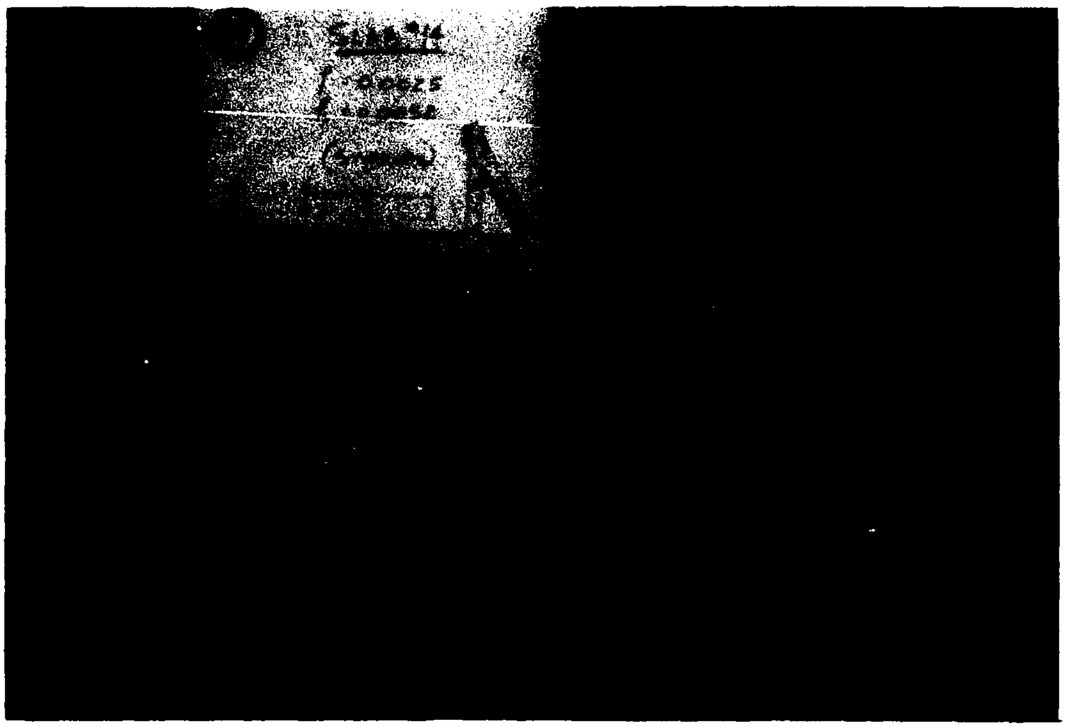


Figure 5.14. Posttest View of Slab No. 14

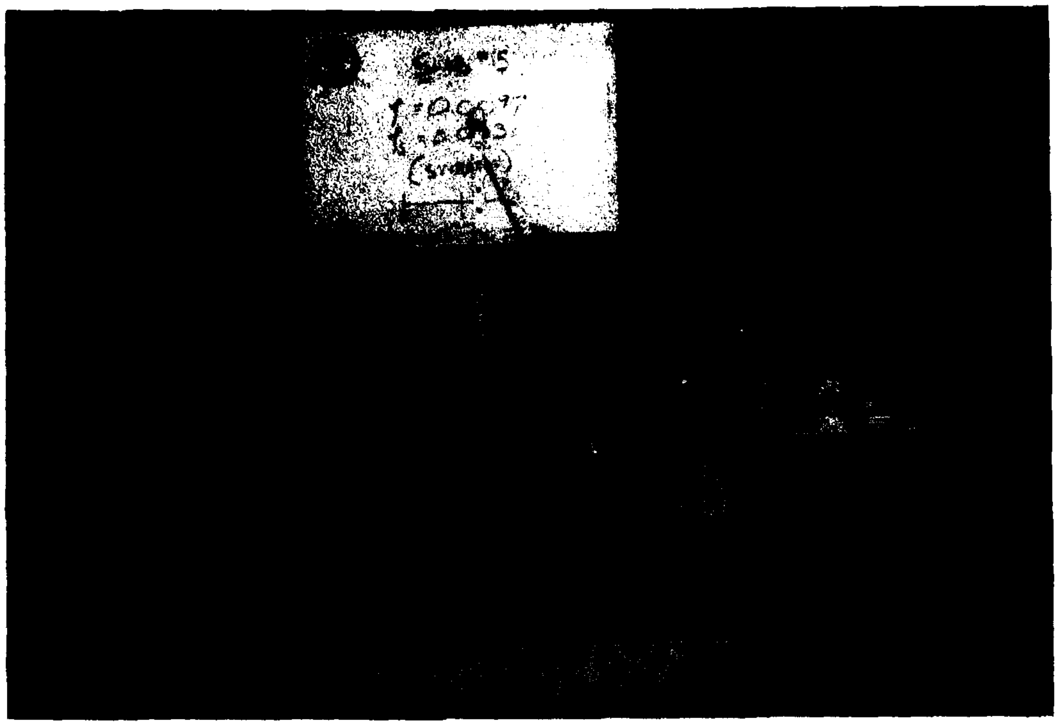


Figure 5.15. Posttest View of Slab No. 15

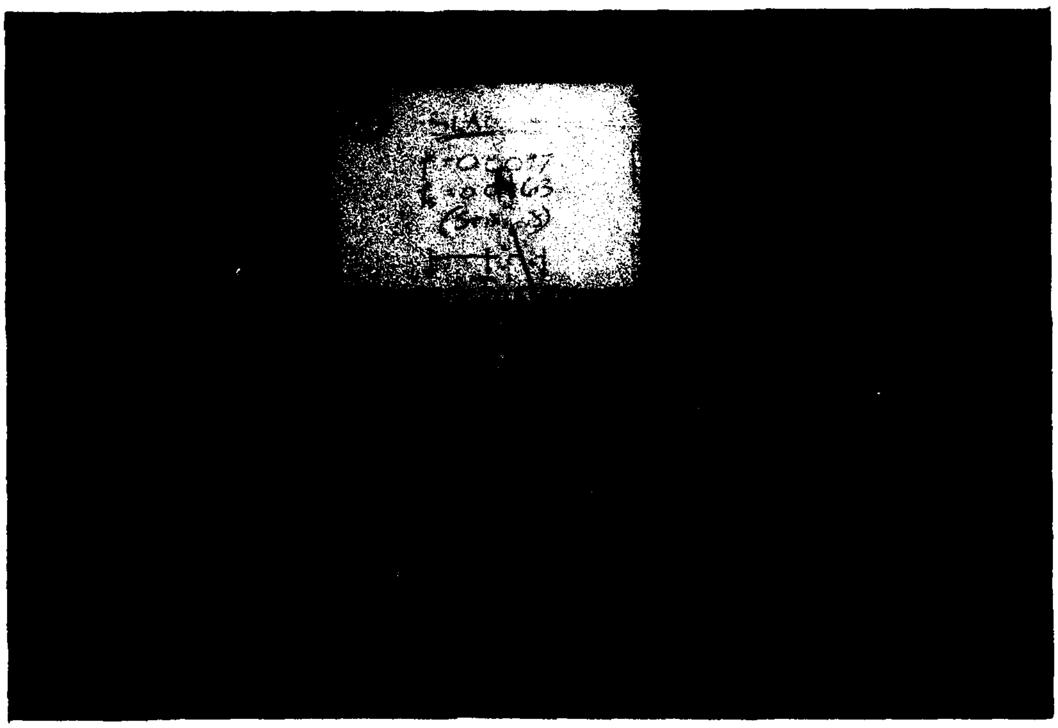


Figure 5.16. Posttest View of Slab No. 16

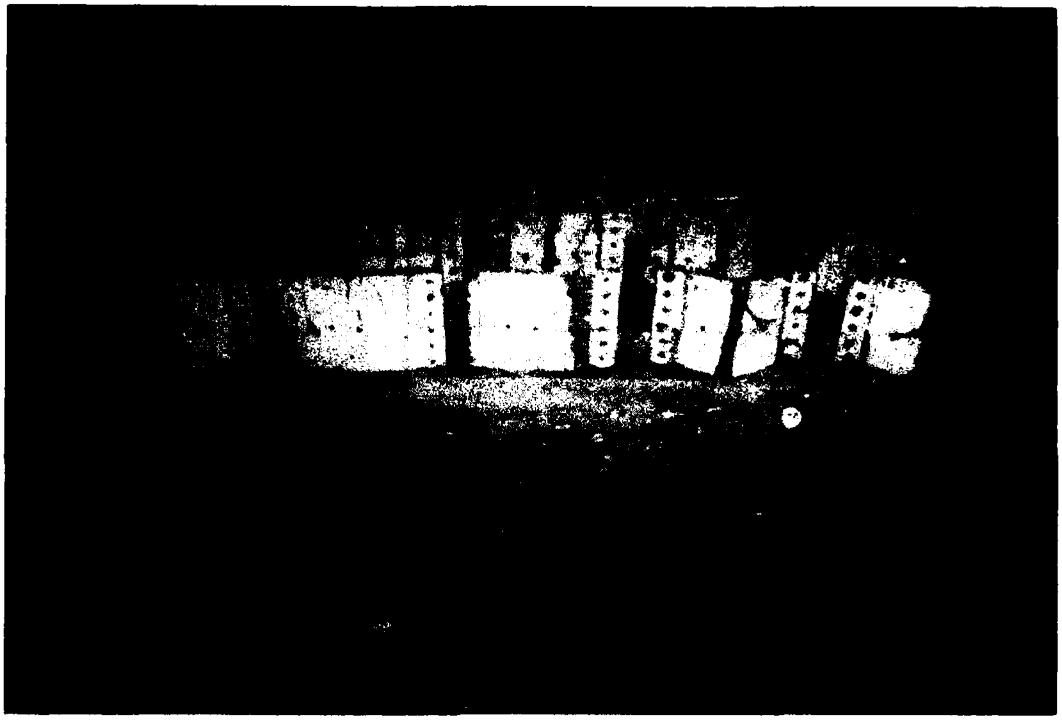


Figure 5.17. Posttest View of Undersurface of Slabs

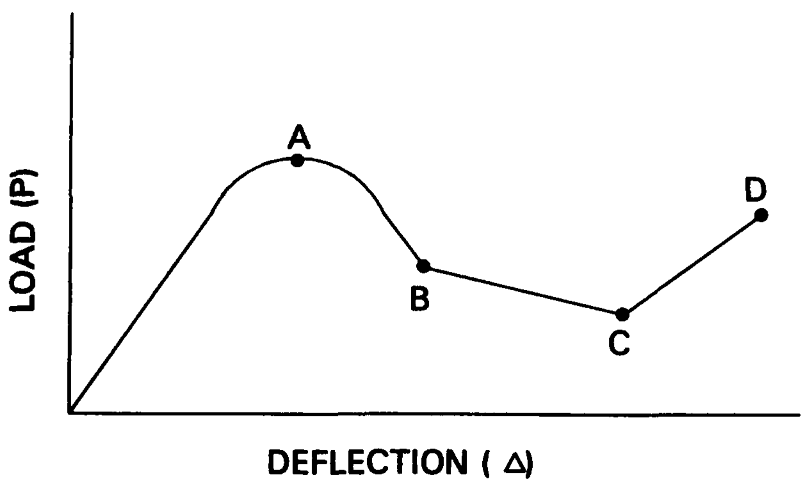


Figure 5.18. General Midspan Load-Deflection Curve

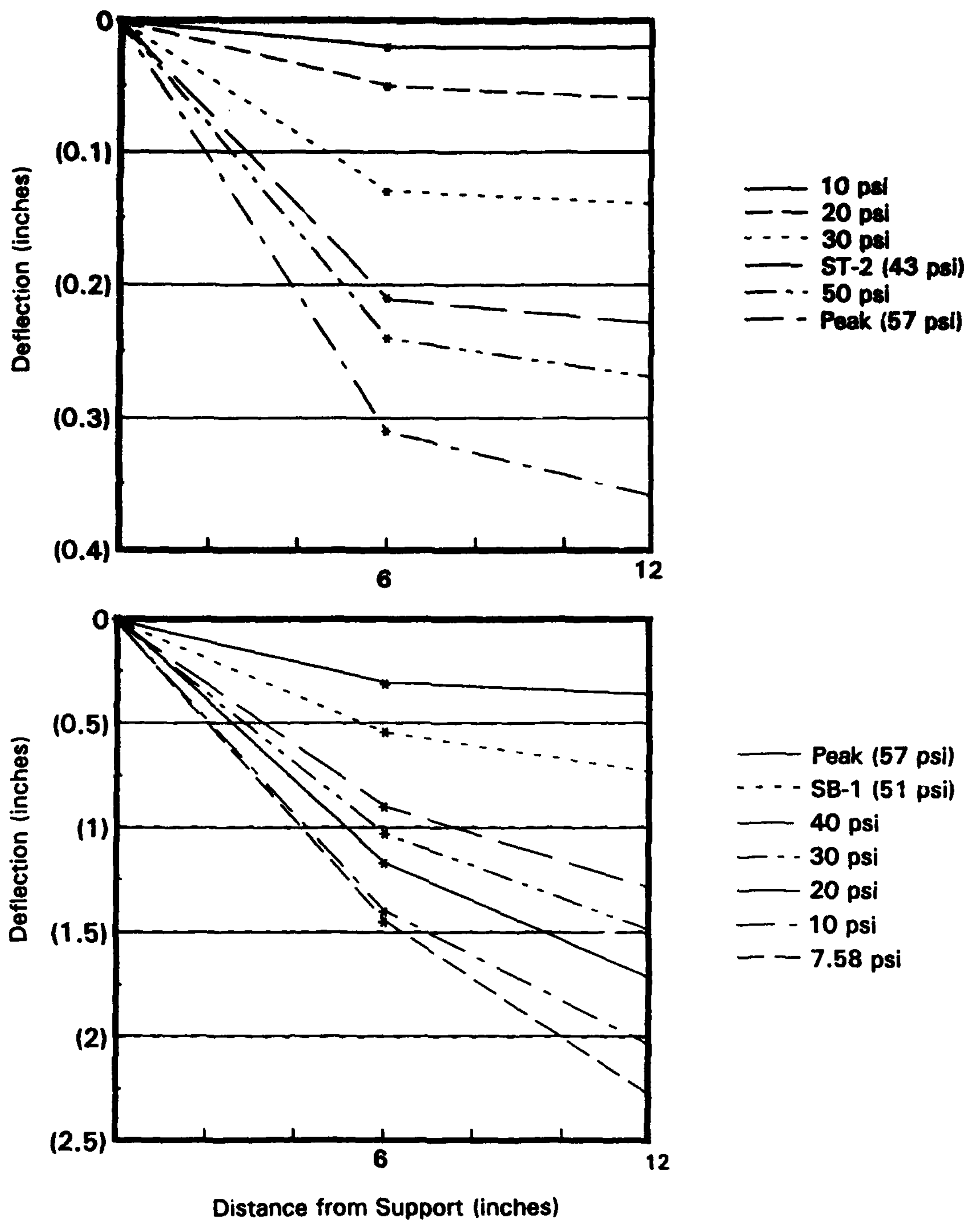


Figure 5.19. Deflection Profile for Slab No. 1

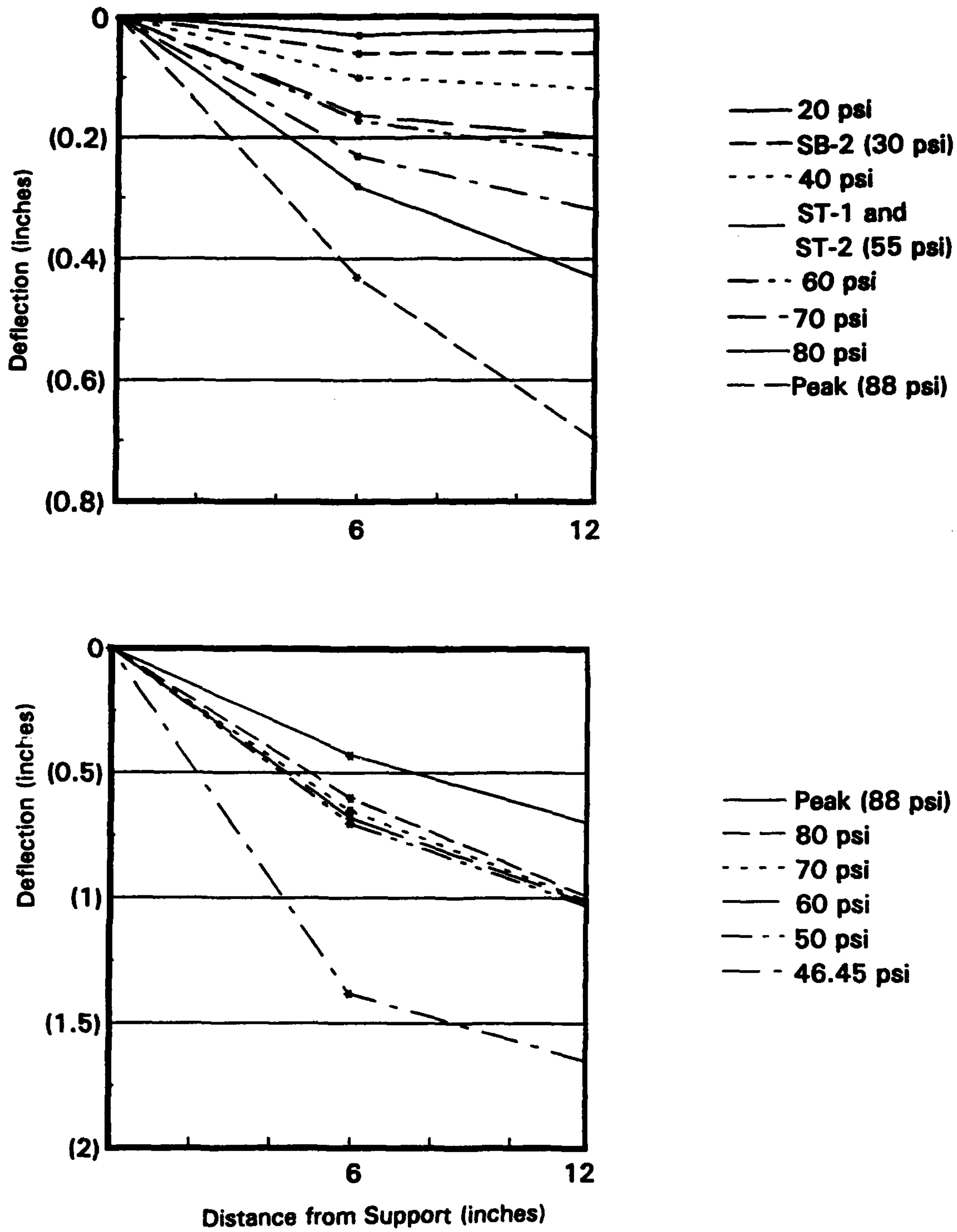


Figure 5.20. Deflection Profile for Slab No. 2

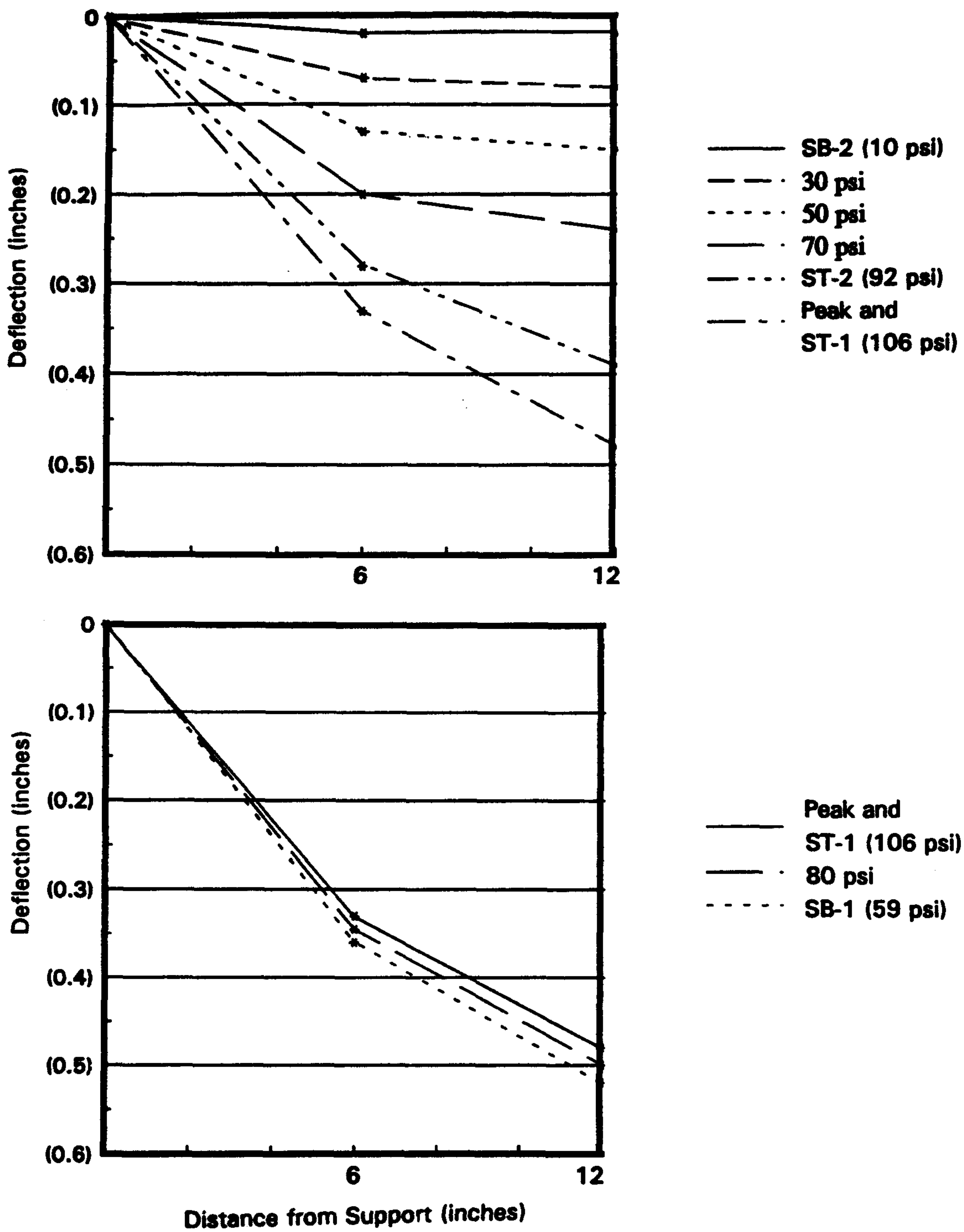


Figure 5.21. Deflection Profile for Slab No. 3

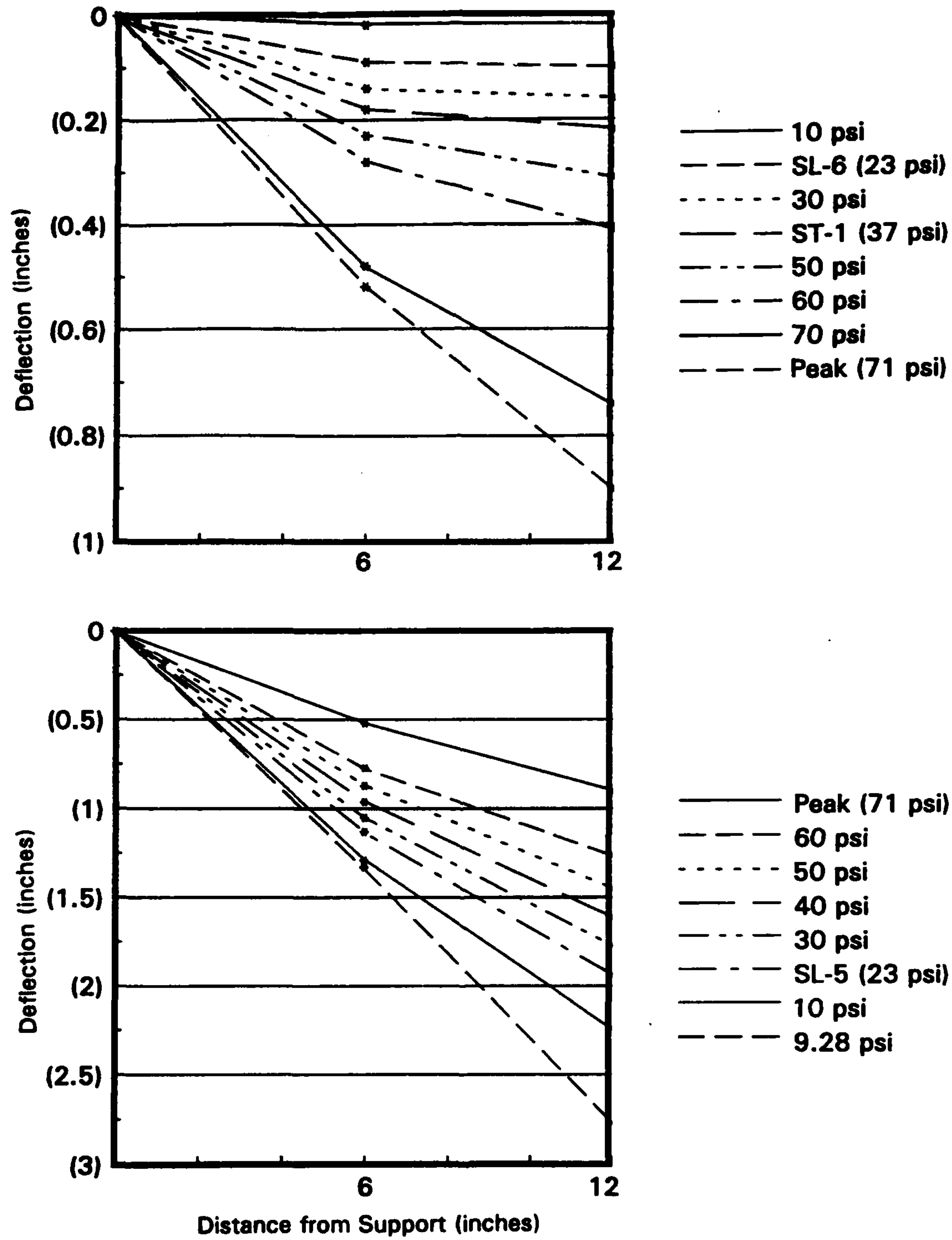


Figure 5.22. Deflection Profile for Slab No. 4

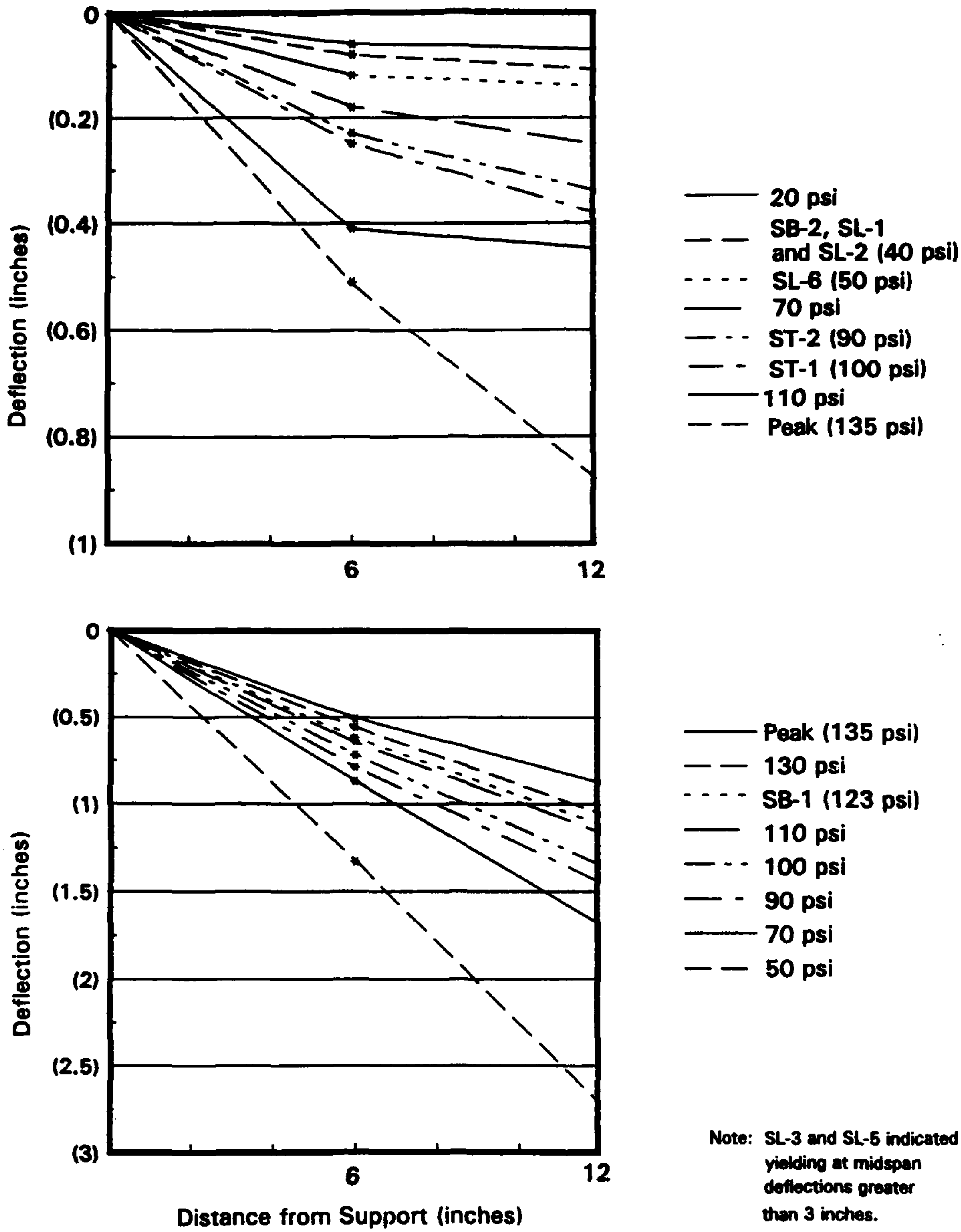


Figure 5.23. Deflection Profile for Slab No. 5

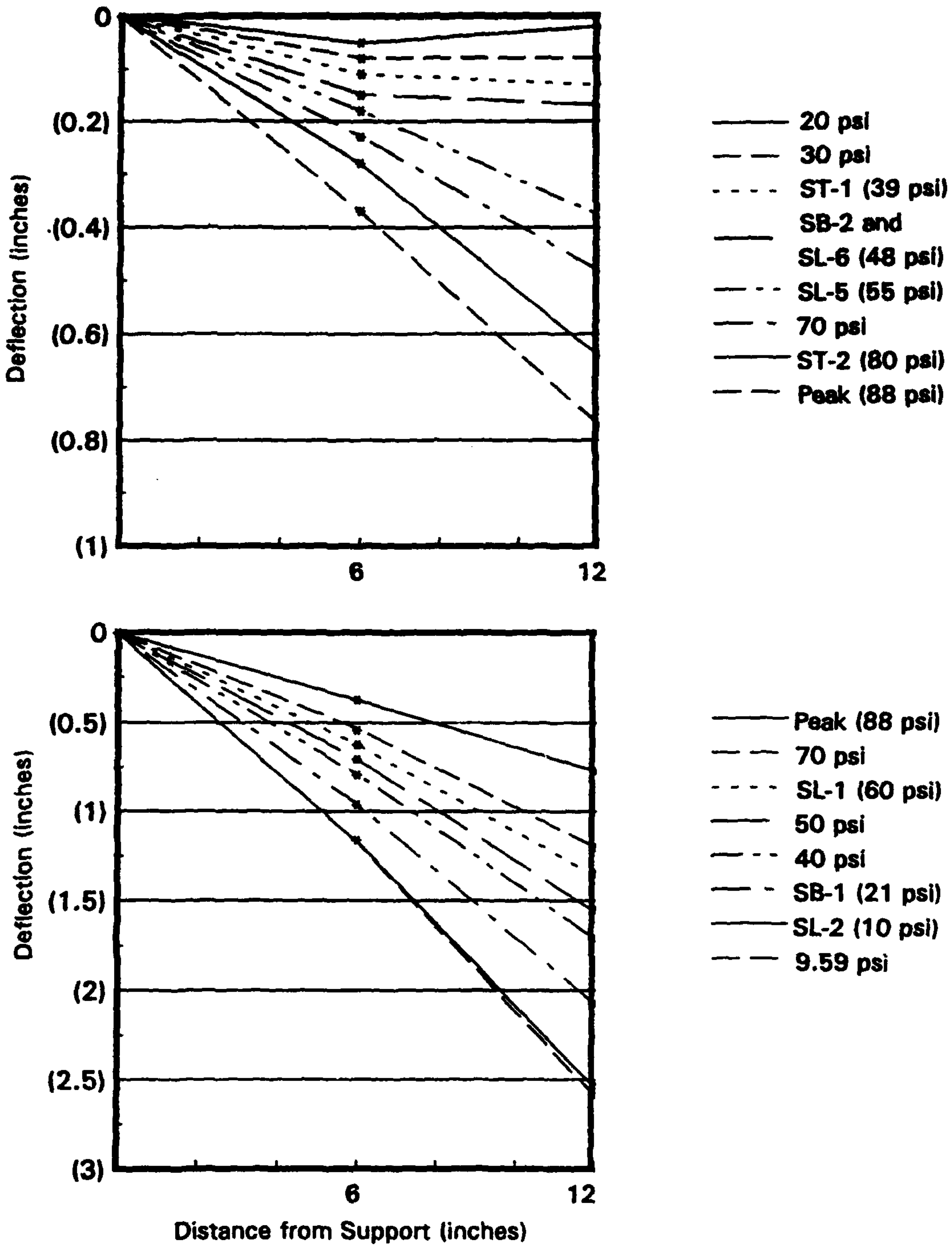


Figure 5.24. Deflection Profile for Slab No. 6

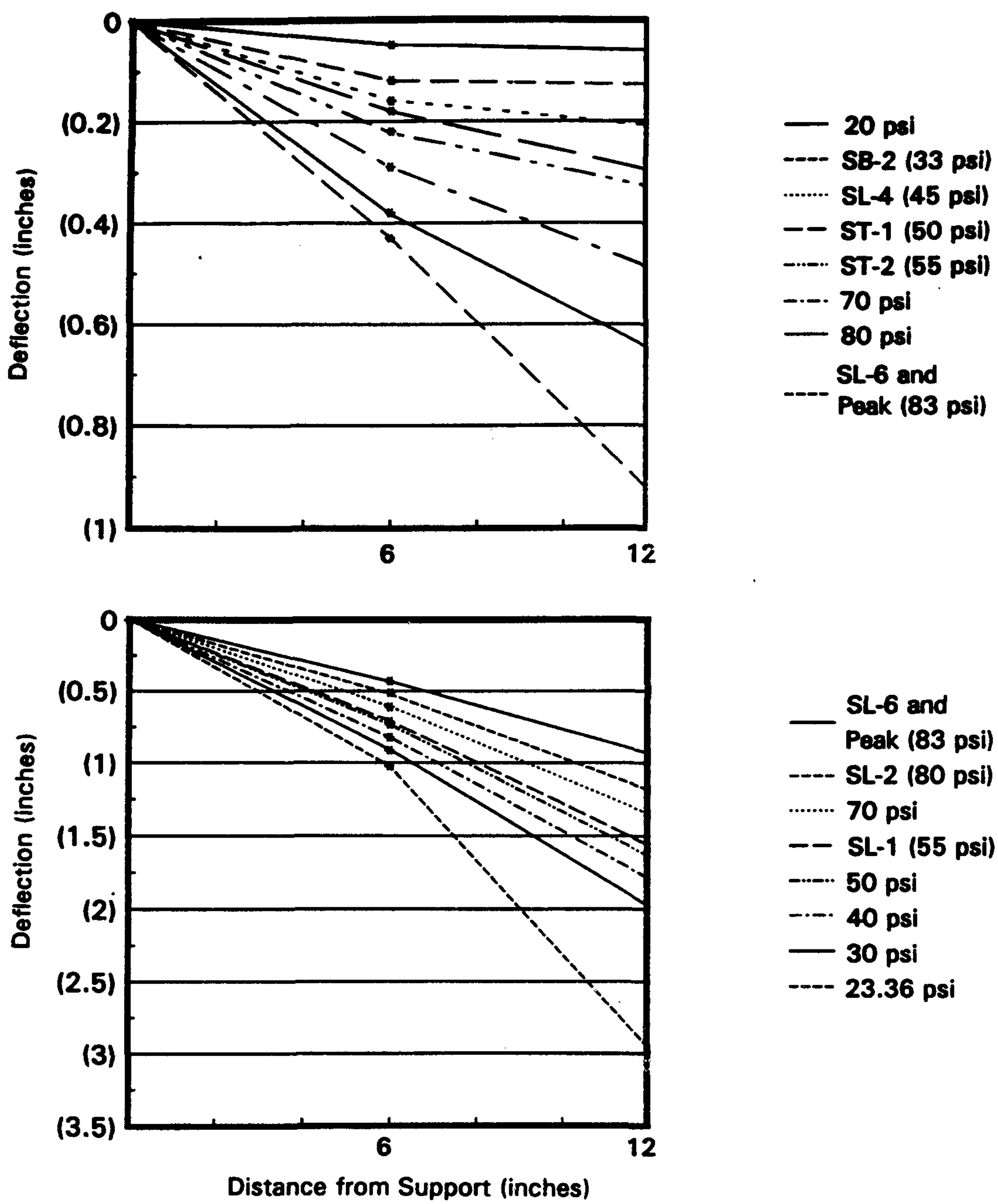


Figure 5.25. Deflection Profile for Slab No. 7

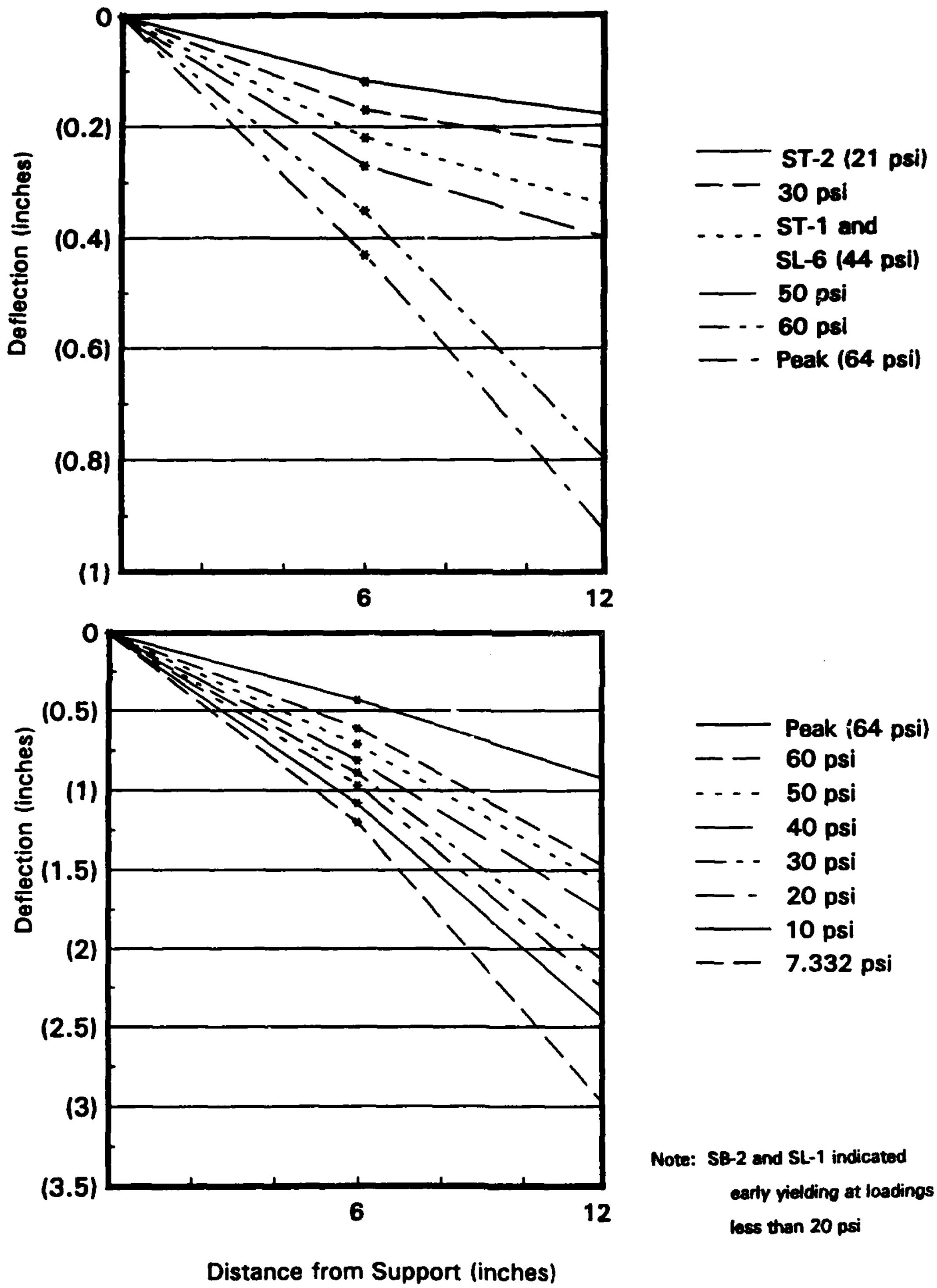


Figure 5.26. Deflection Profile for Slab No. 8

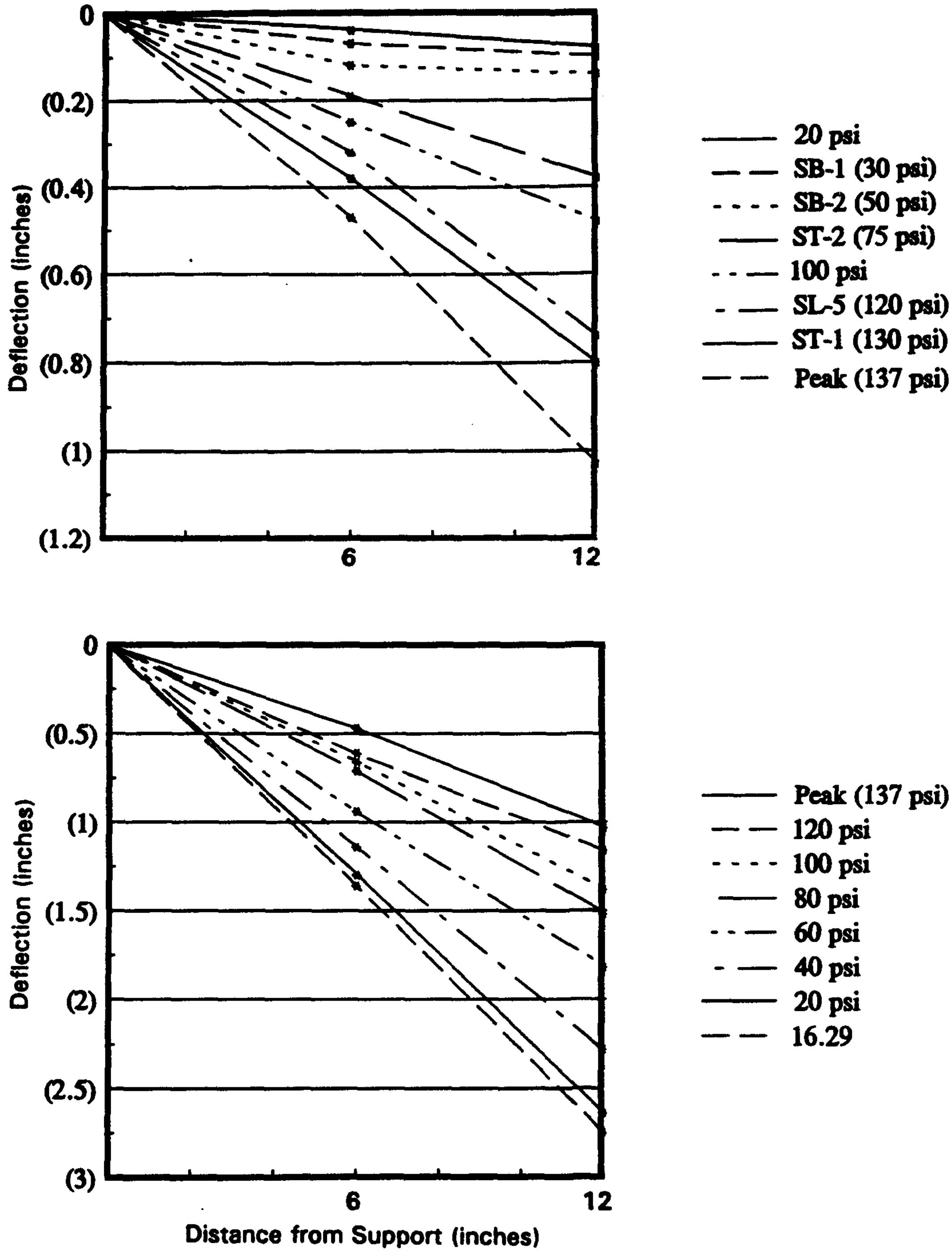


Figure 5.27. Deflection Profile for Slab No. 9

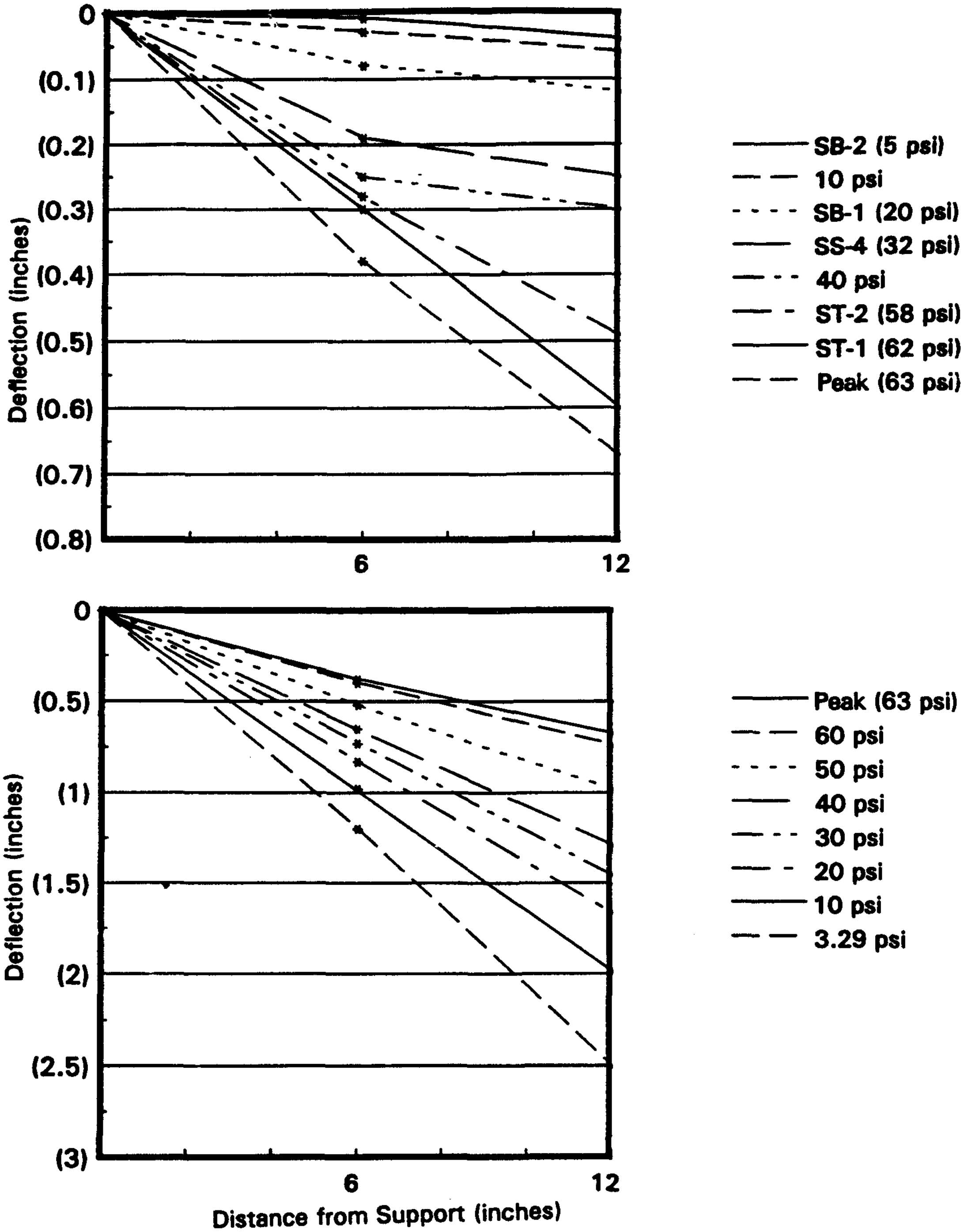


Figure 5.28. Deflection Profile for Slab No. 10

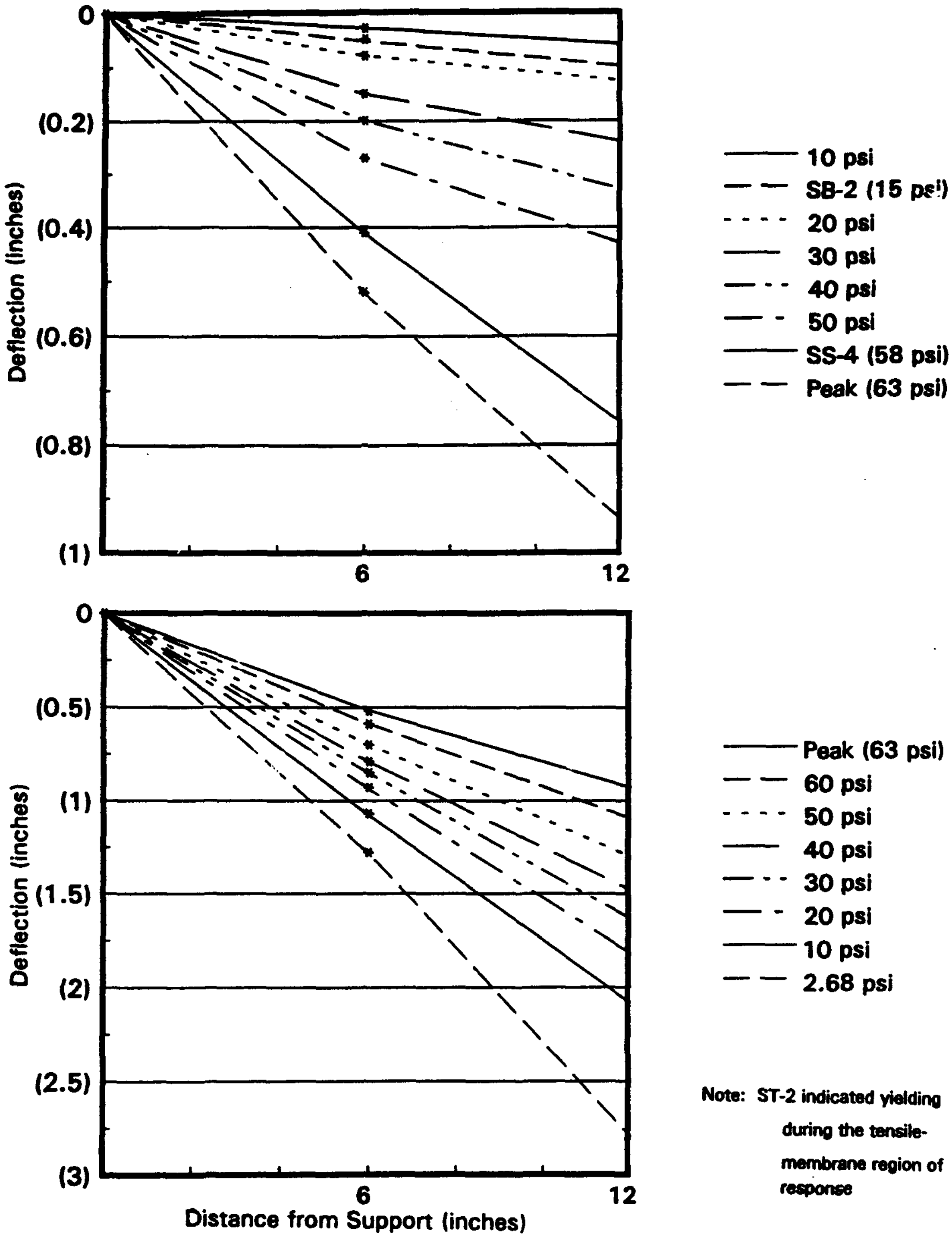


Figure 5.29. Deflection Profile for Slab No. 11

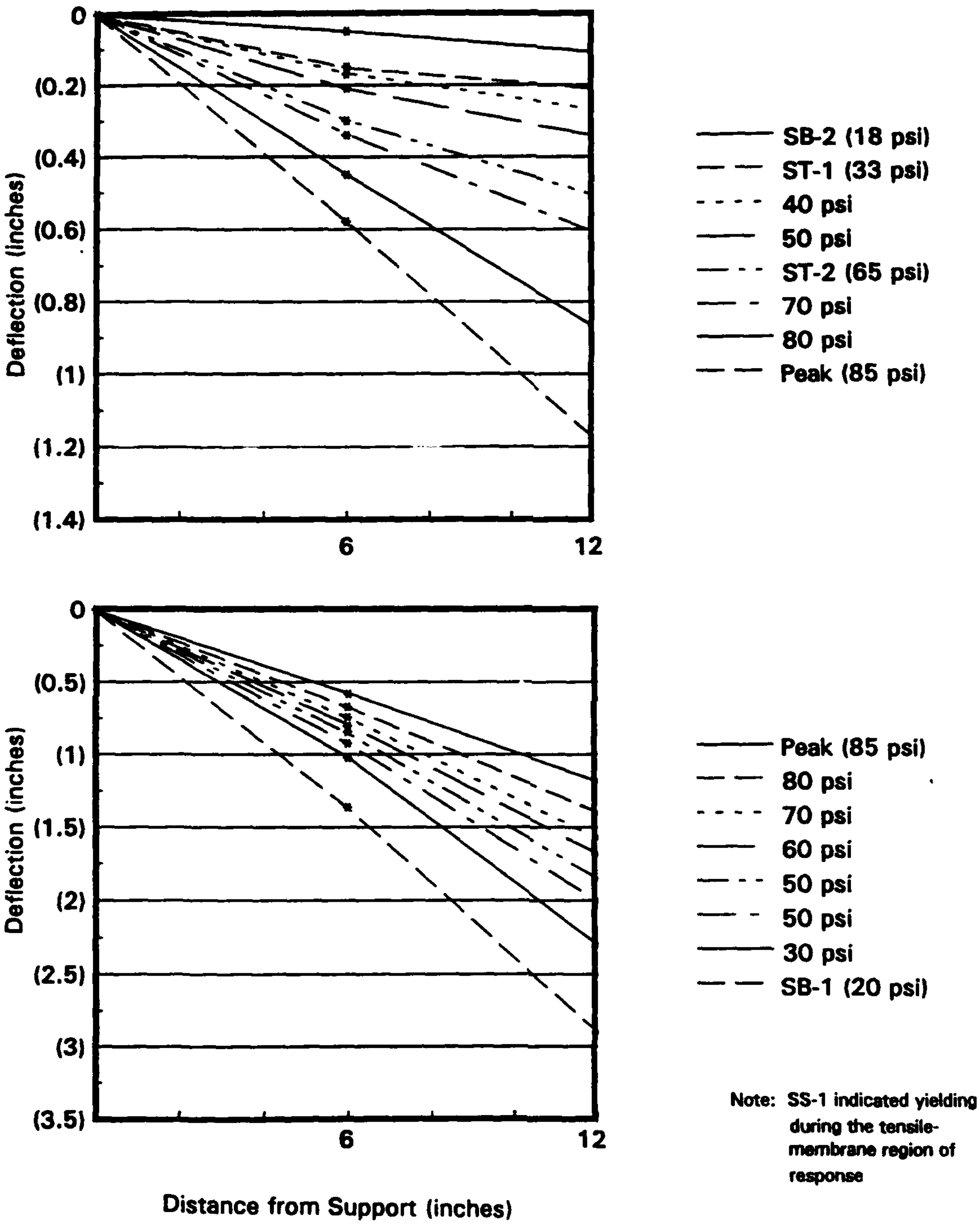


Figure 5.30. Deflection Profile for Slab No. 12

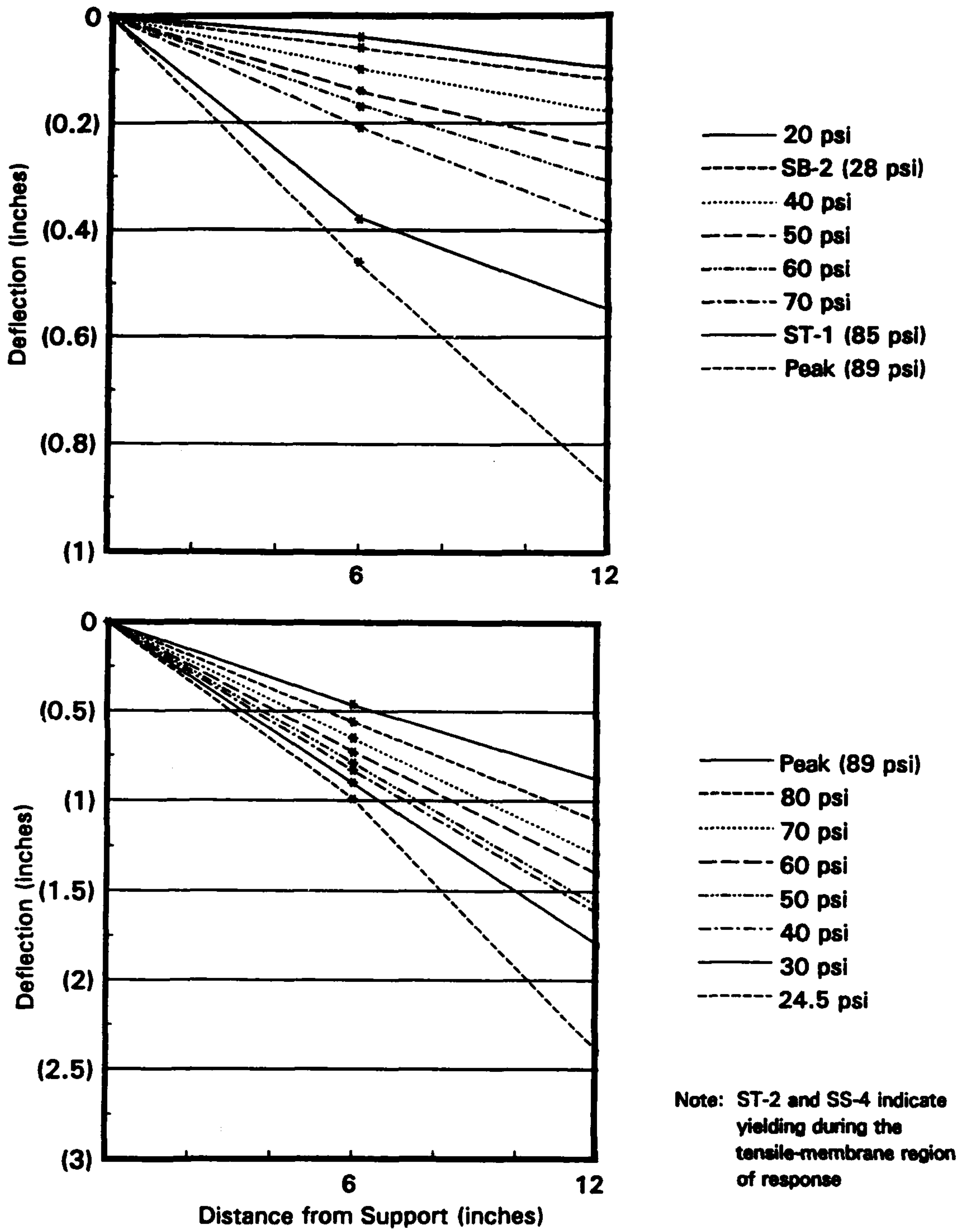


Figure 5.31. Deflection Profile for Slab No. 13

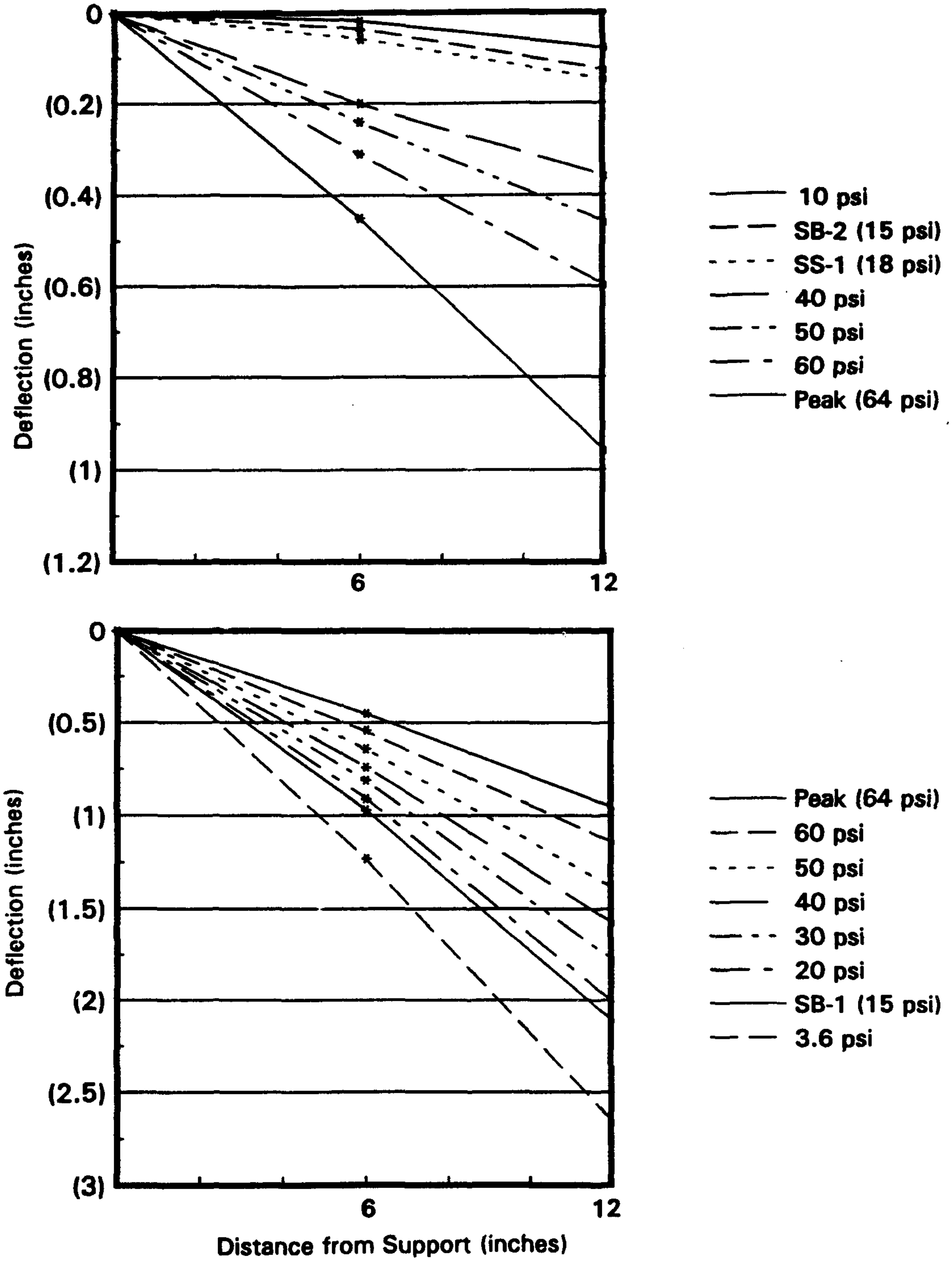


Figure 5.32. Deflection Profile for Slab No. 14

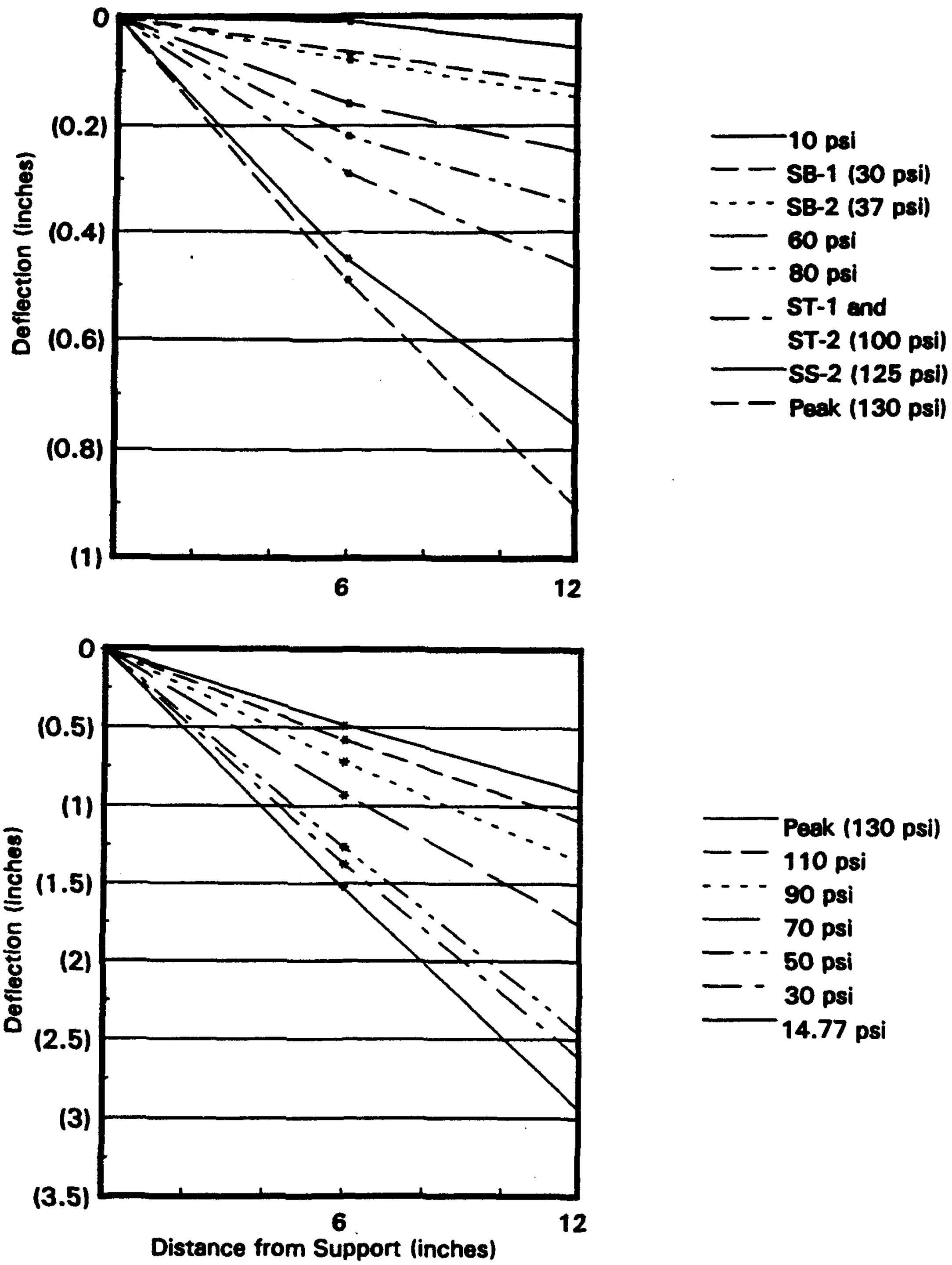


Figure 5.33. Deflection Profile for Slab No. 15

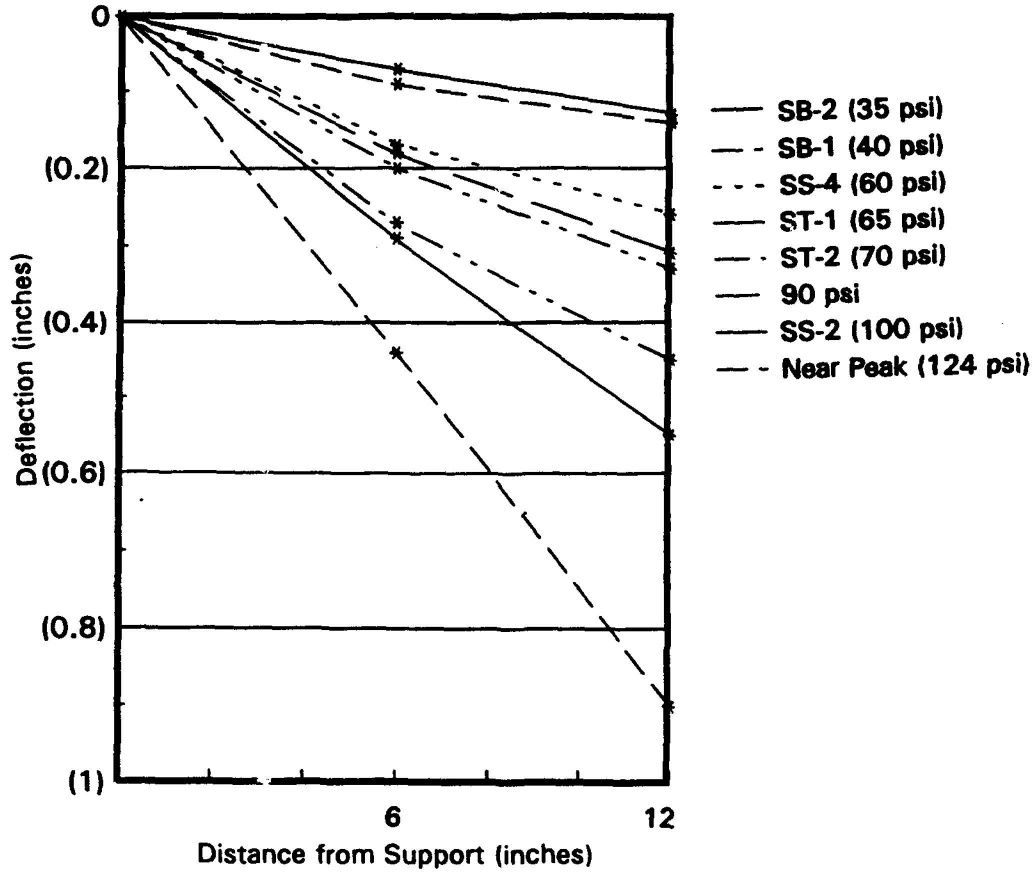


Figure 5.34. Deflection Profile for Slab No. 16

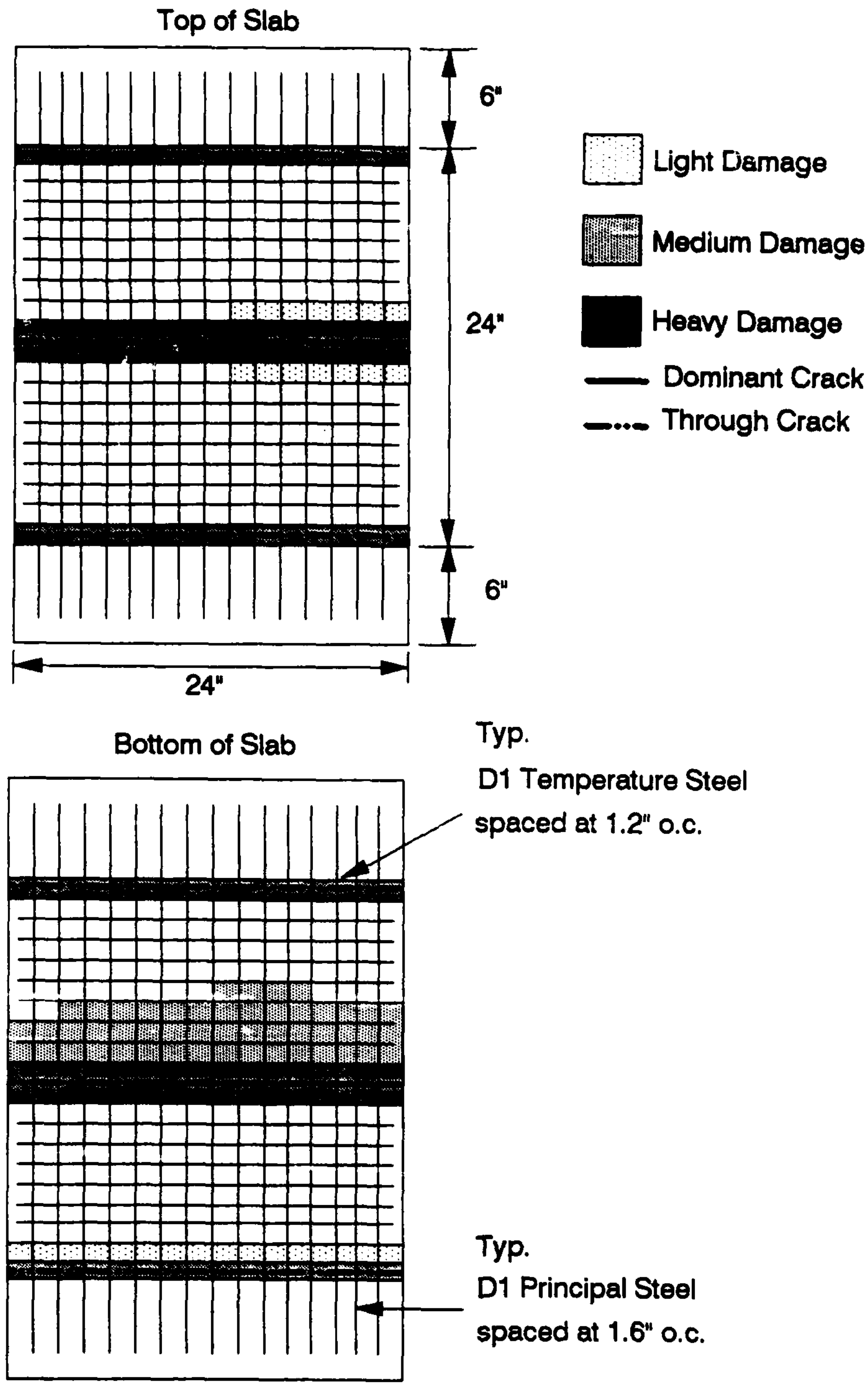


Figure 5.35. Damage Survey of Slab No. 1

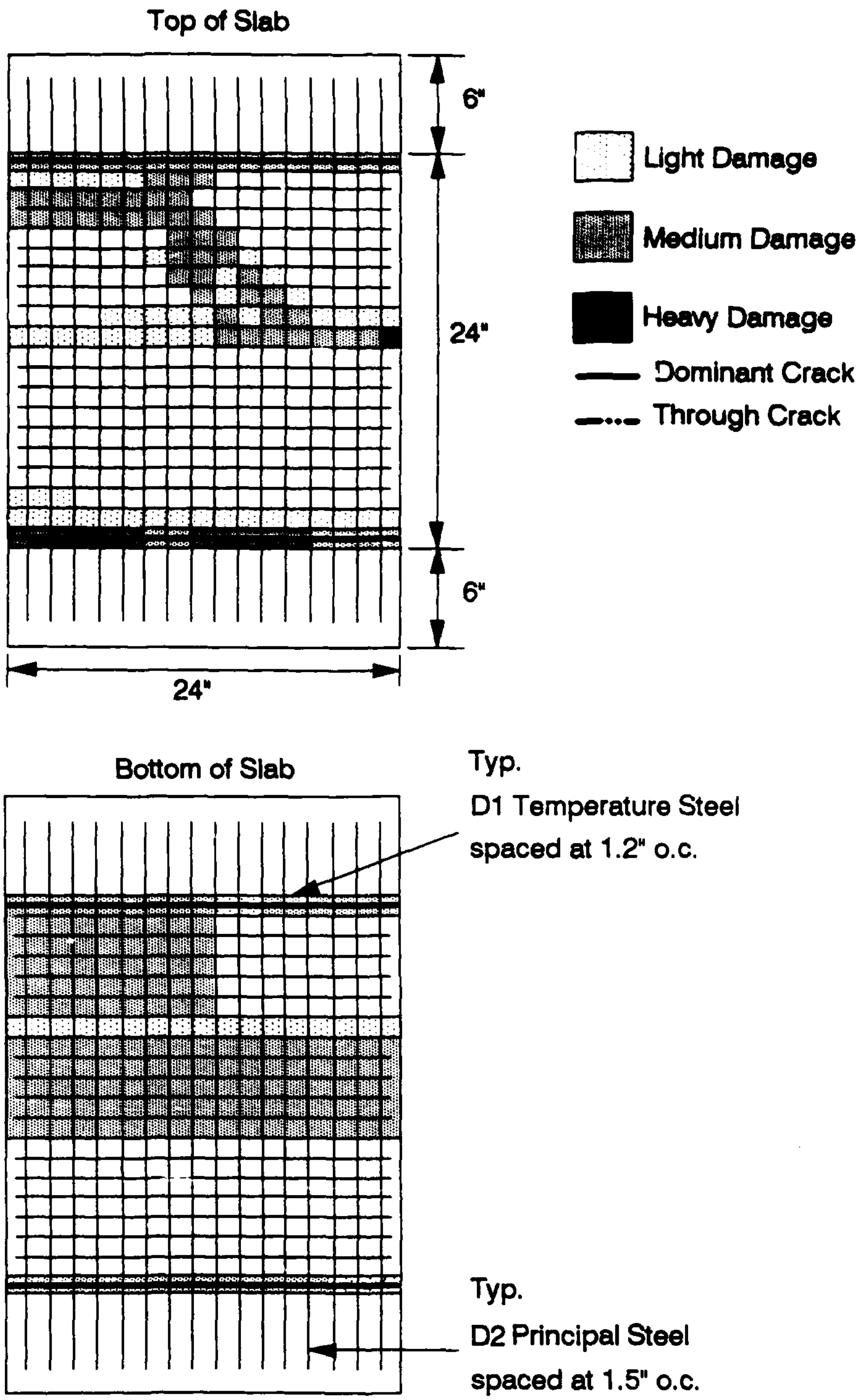


Figure 5.36. Damage Survey of Slab No. 2

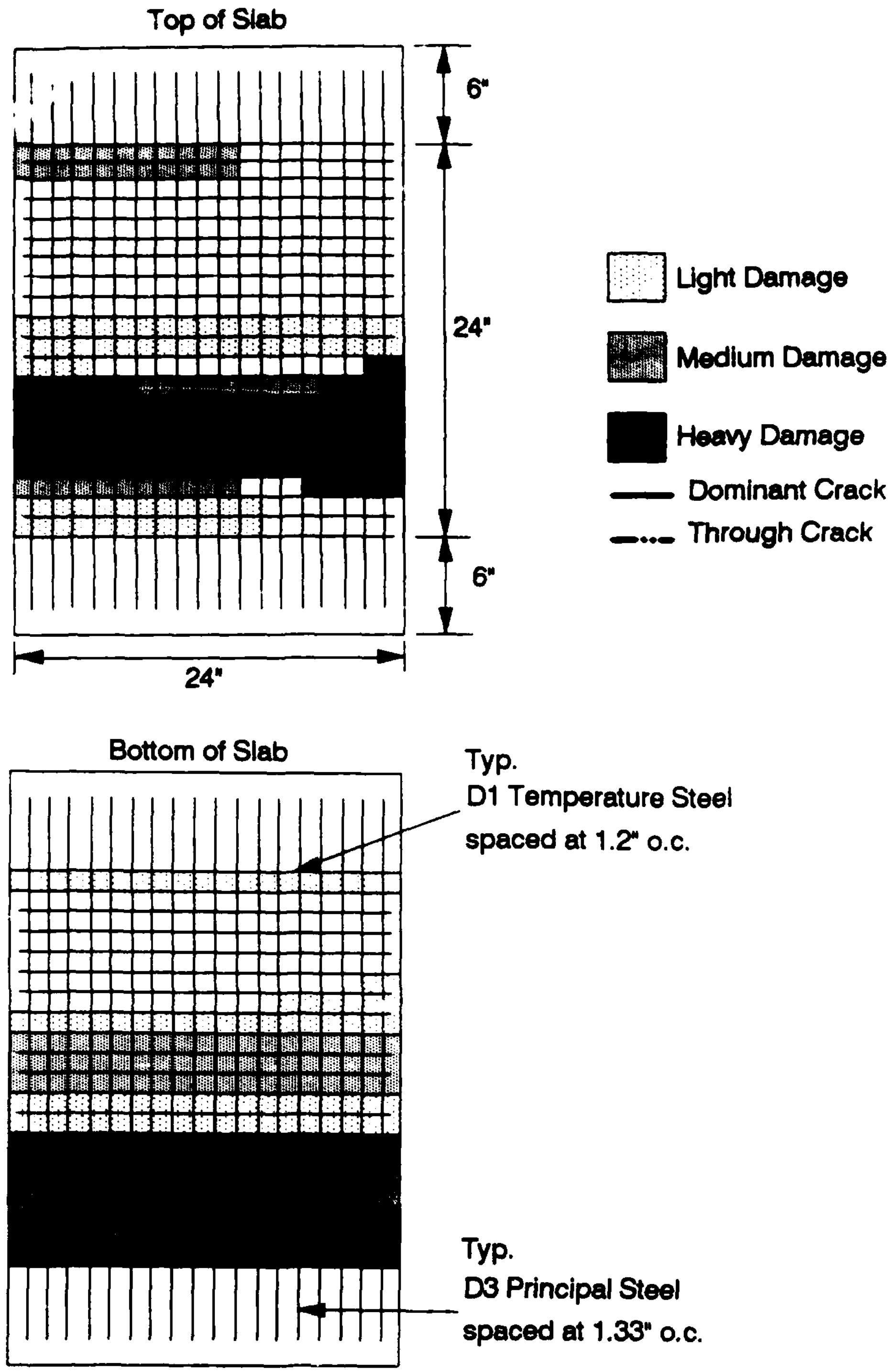


Figure 5.37. Damage Survey of Slab No. 3

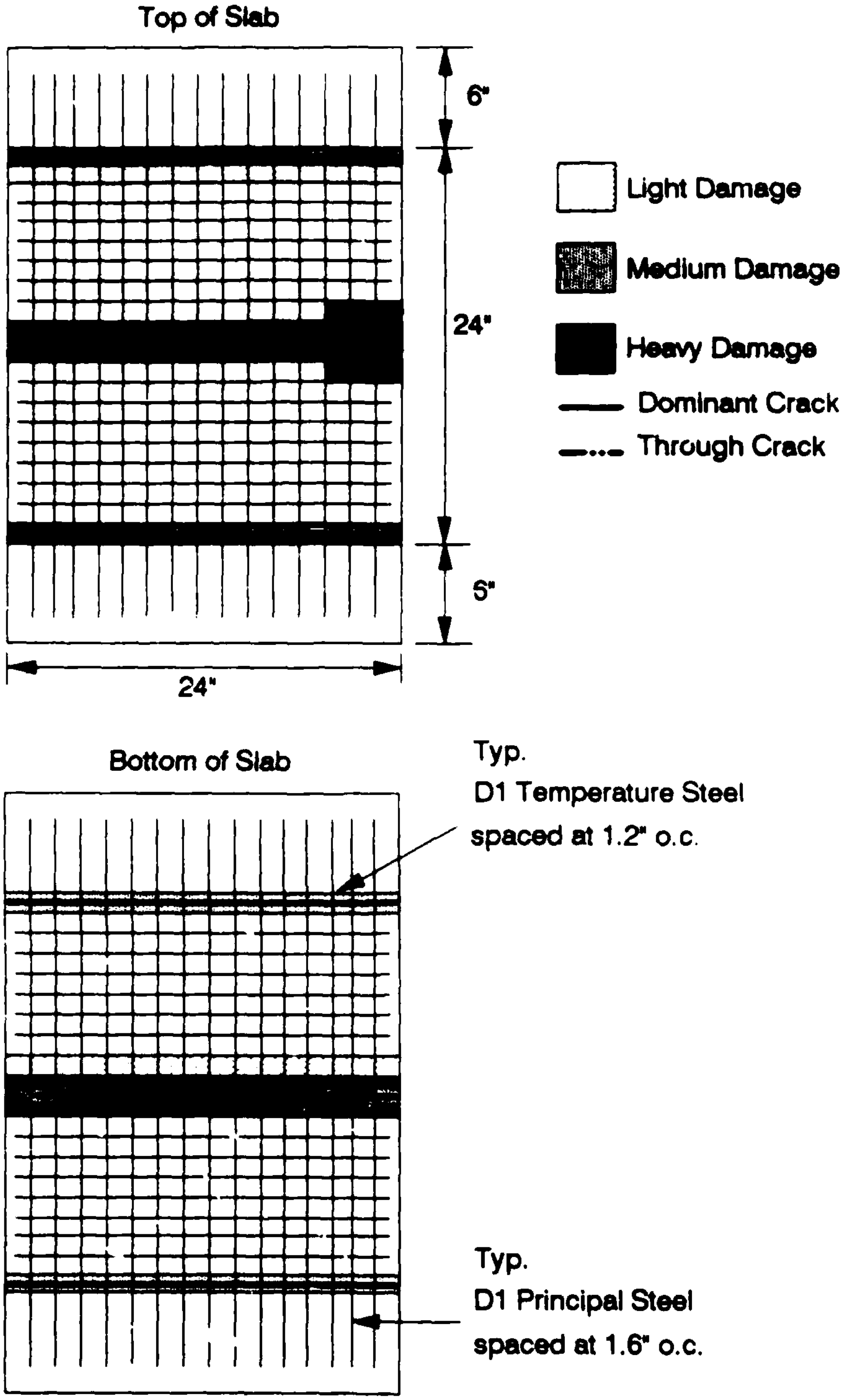


Figure 5.38. Damage Survey of Slab No. 4

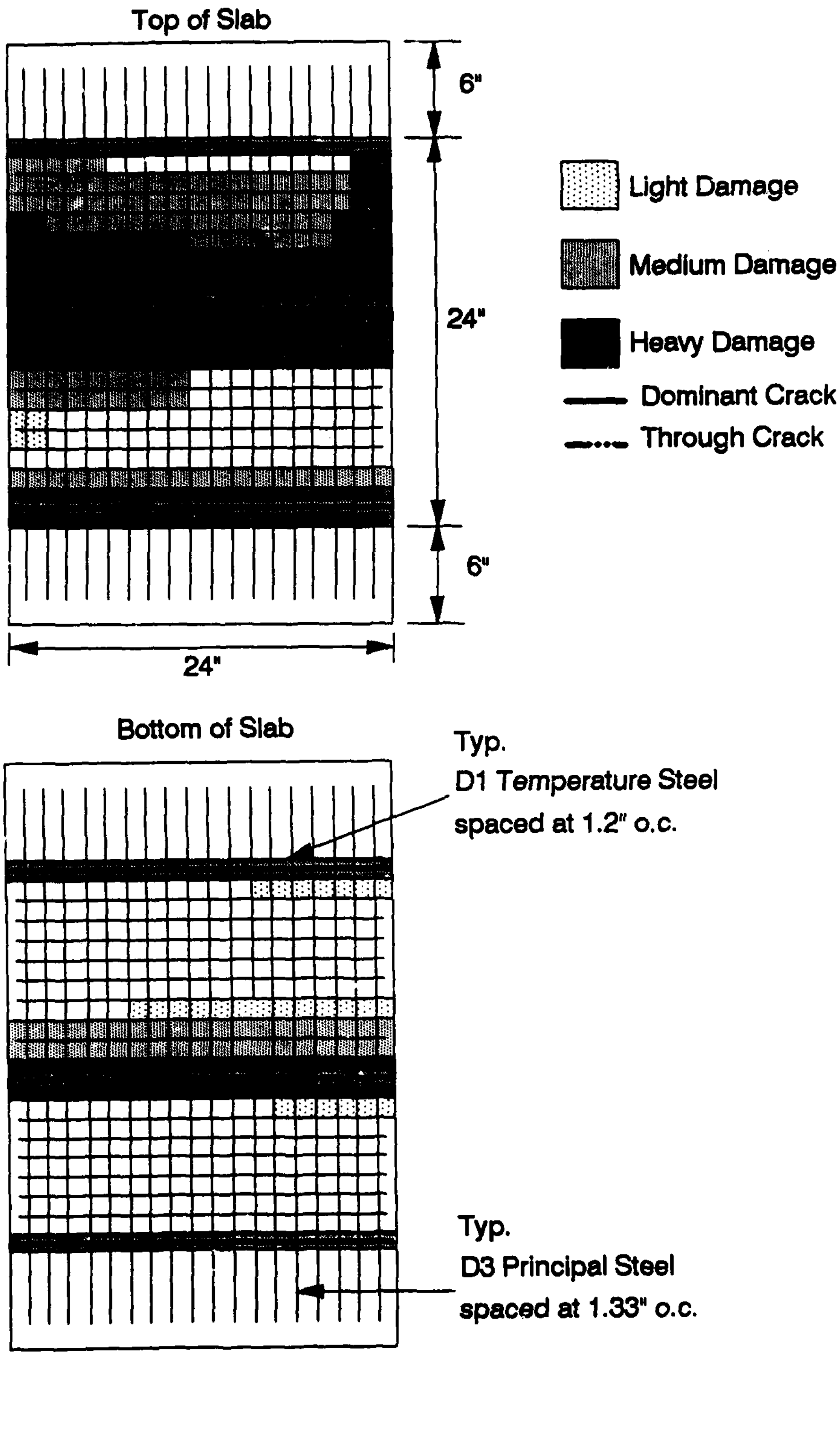


Figure 5.39. Damage Survey of Slab No. 5

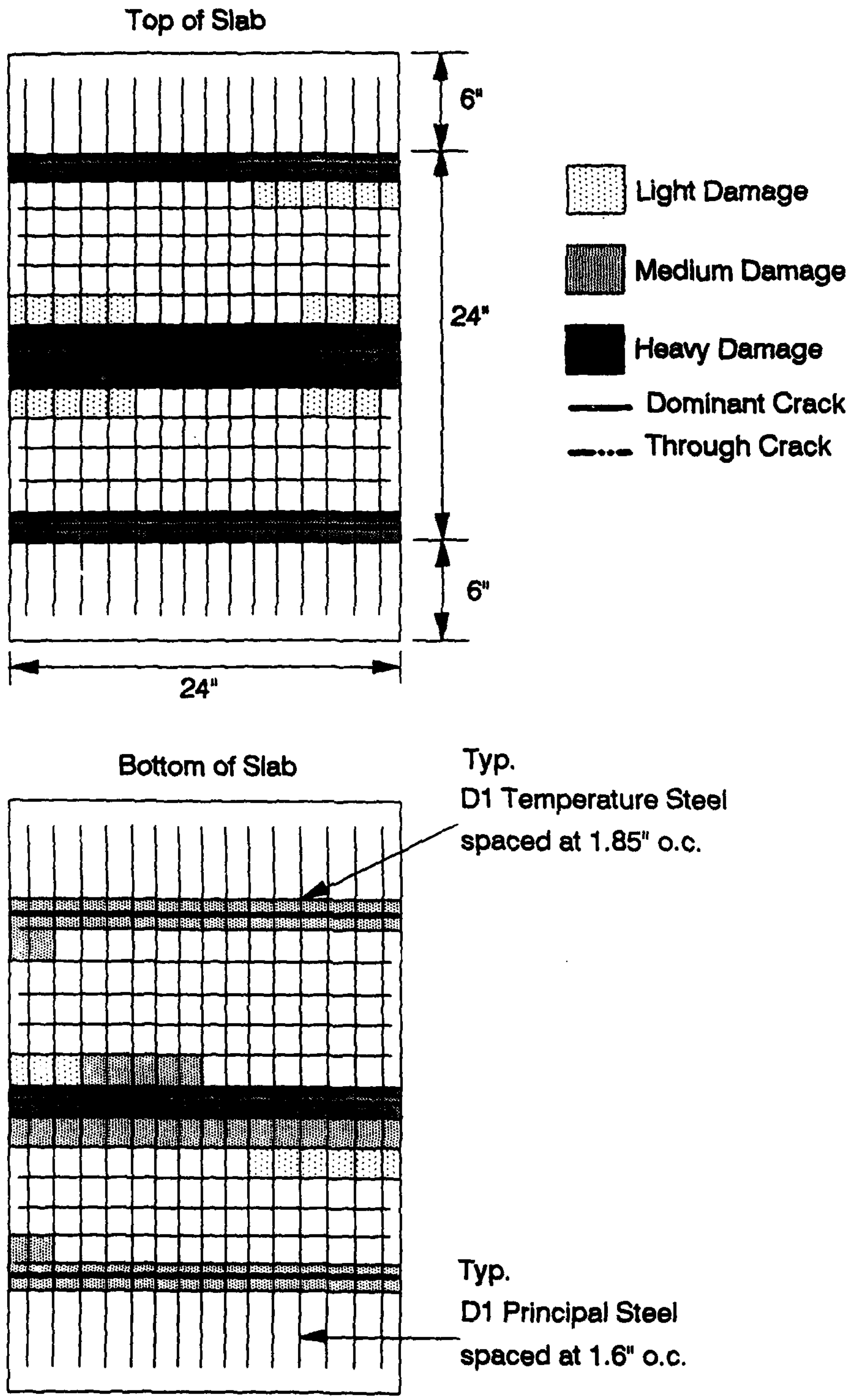


Figure 5.40. Damage Survey of Slab No. 6

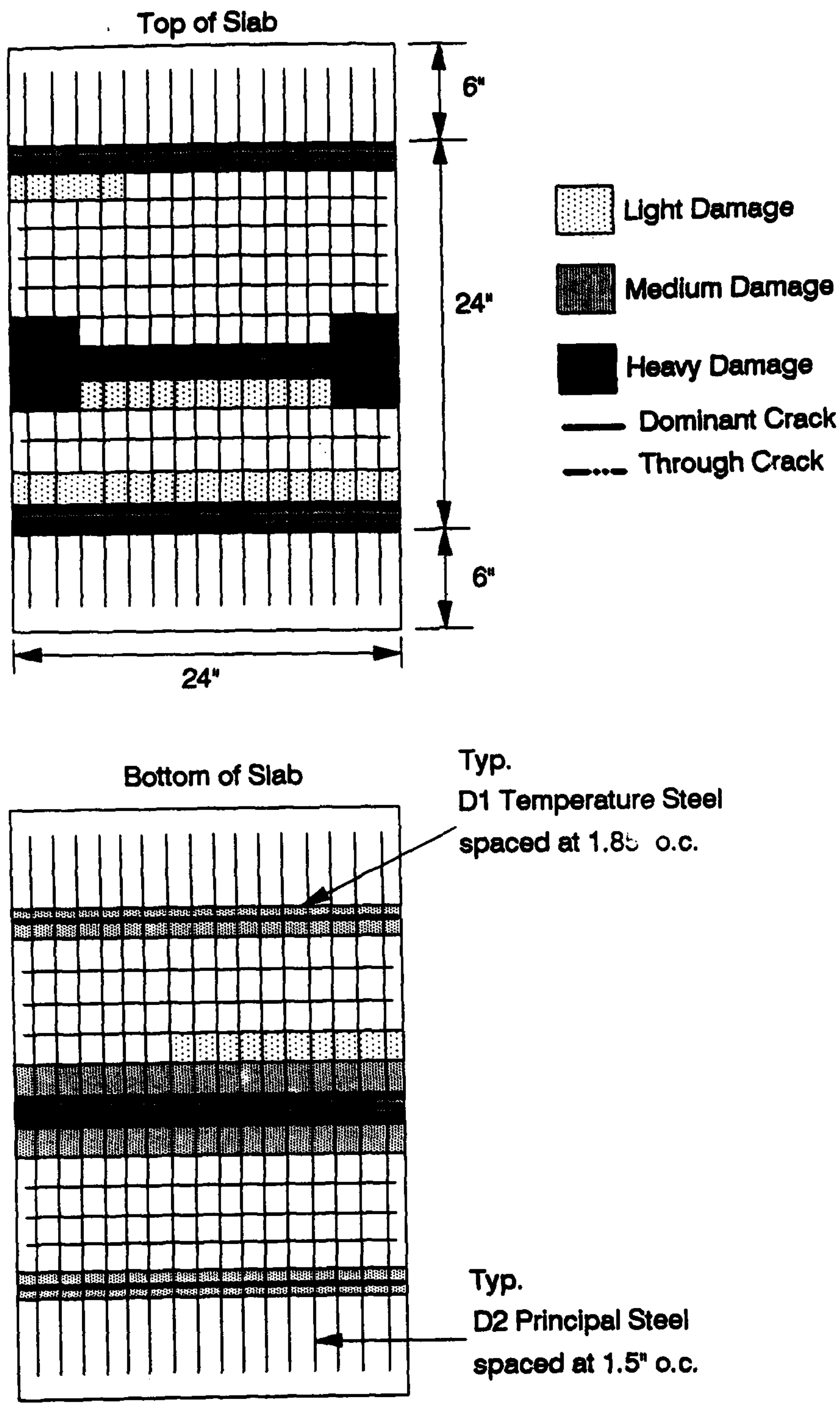


Figure 5.41. Damage Survey of Slab No. 7

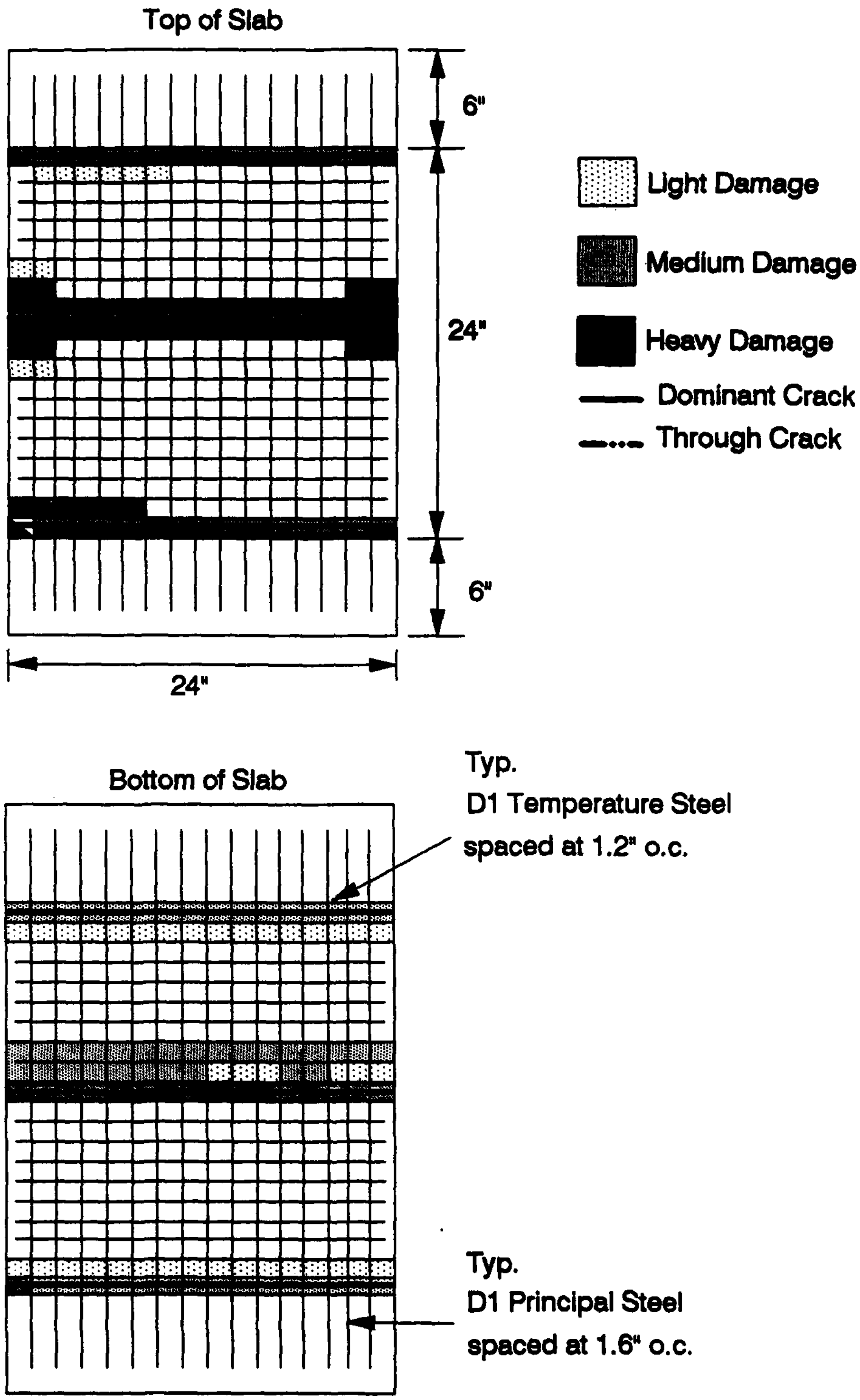


Figure 5.42. Damage Survey of Slab No. 8

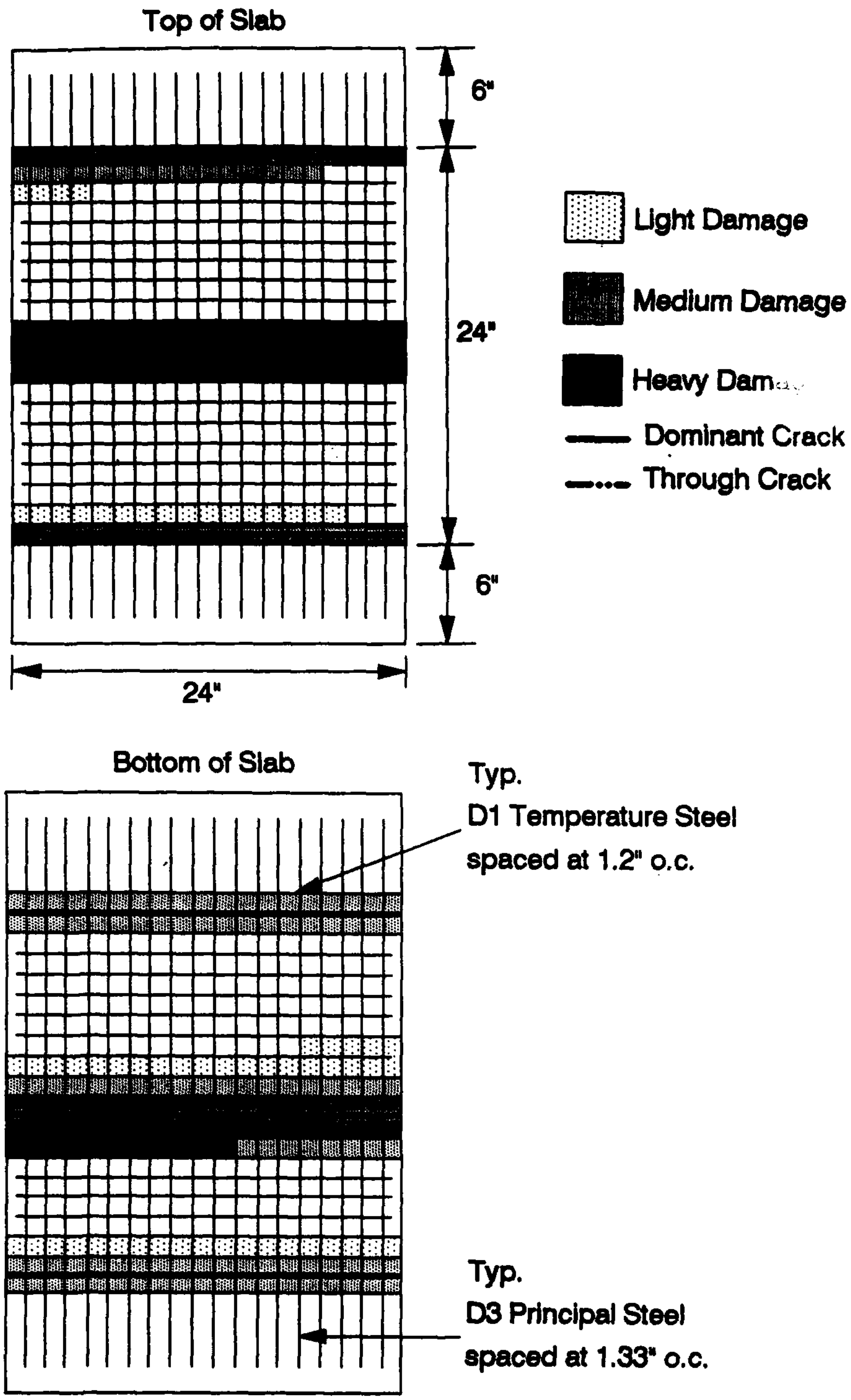


Figure 5.43. Damage Survey of Slab No. 9

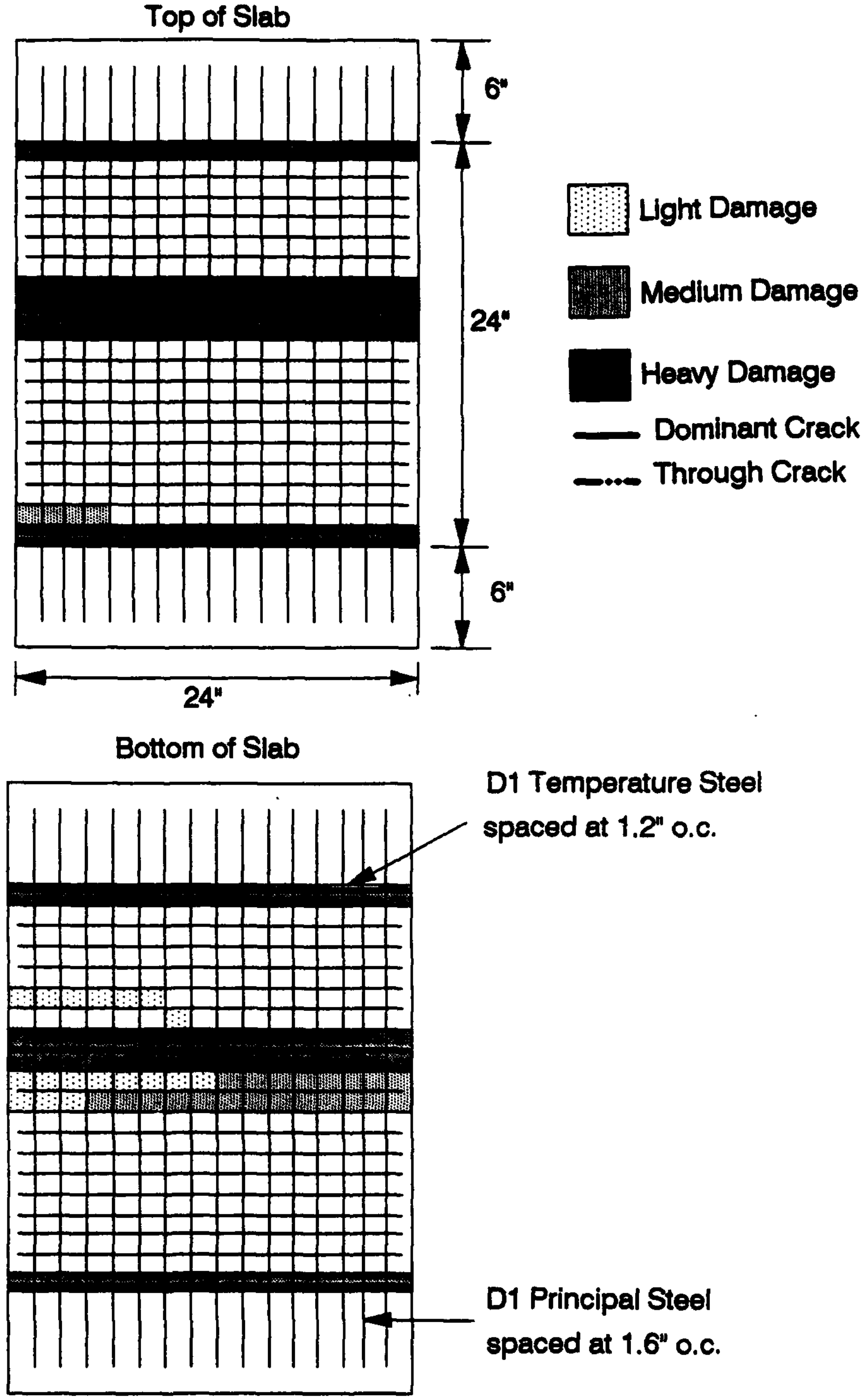


Figure 5.44. Damage Survey of Slab No. 10

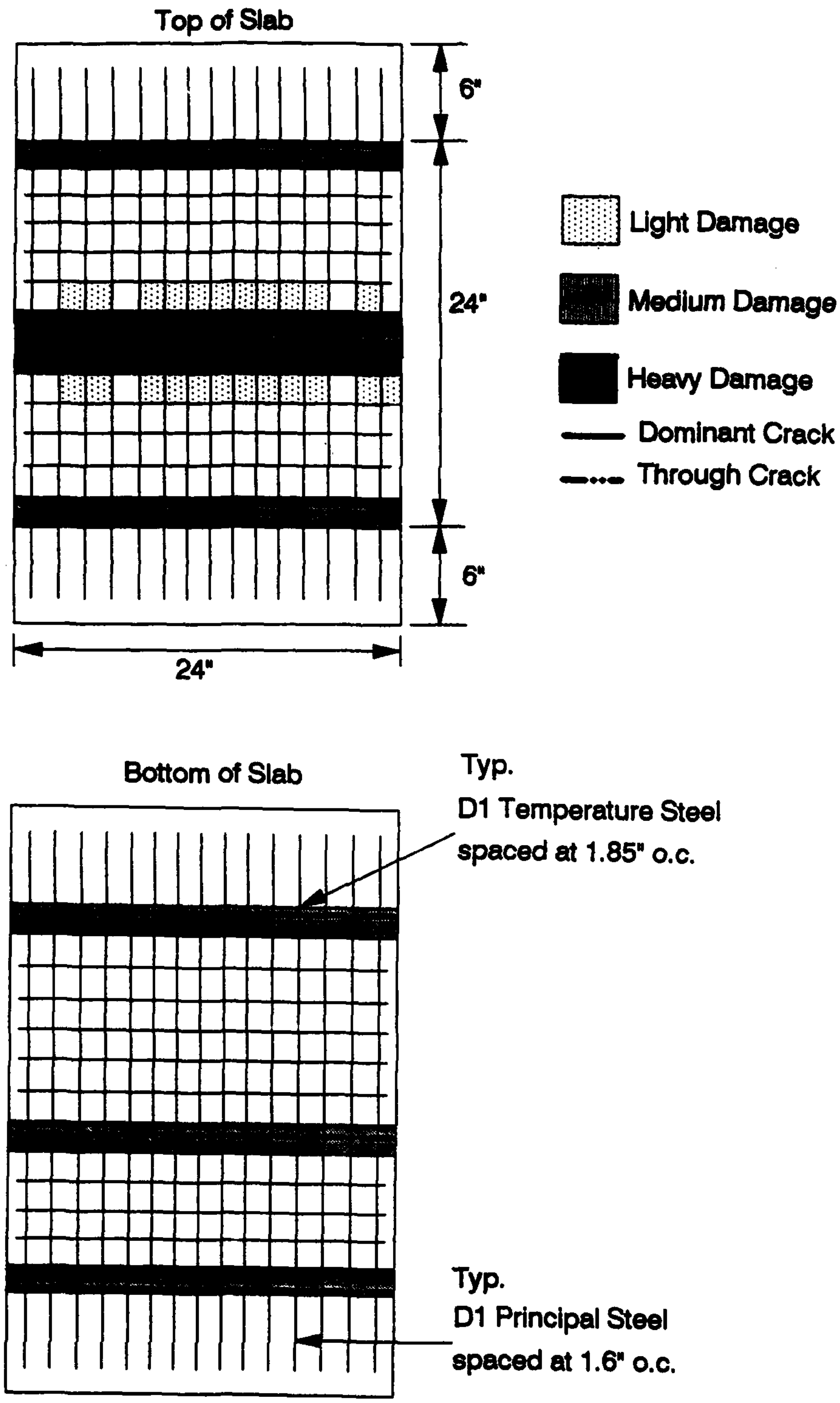


Figure 5.45. Damage Survey of Slab No. 11

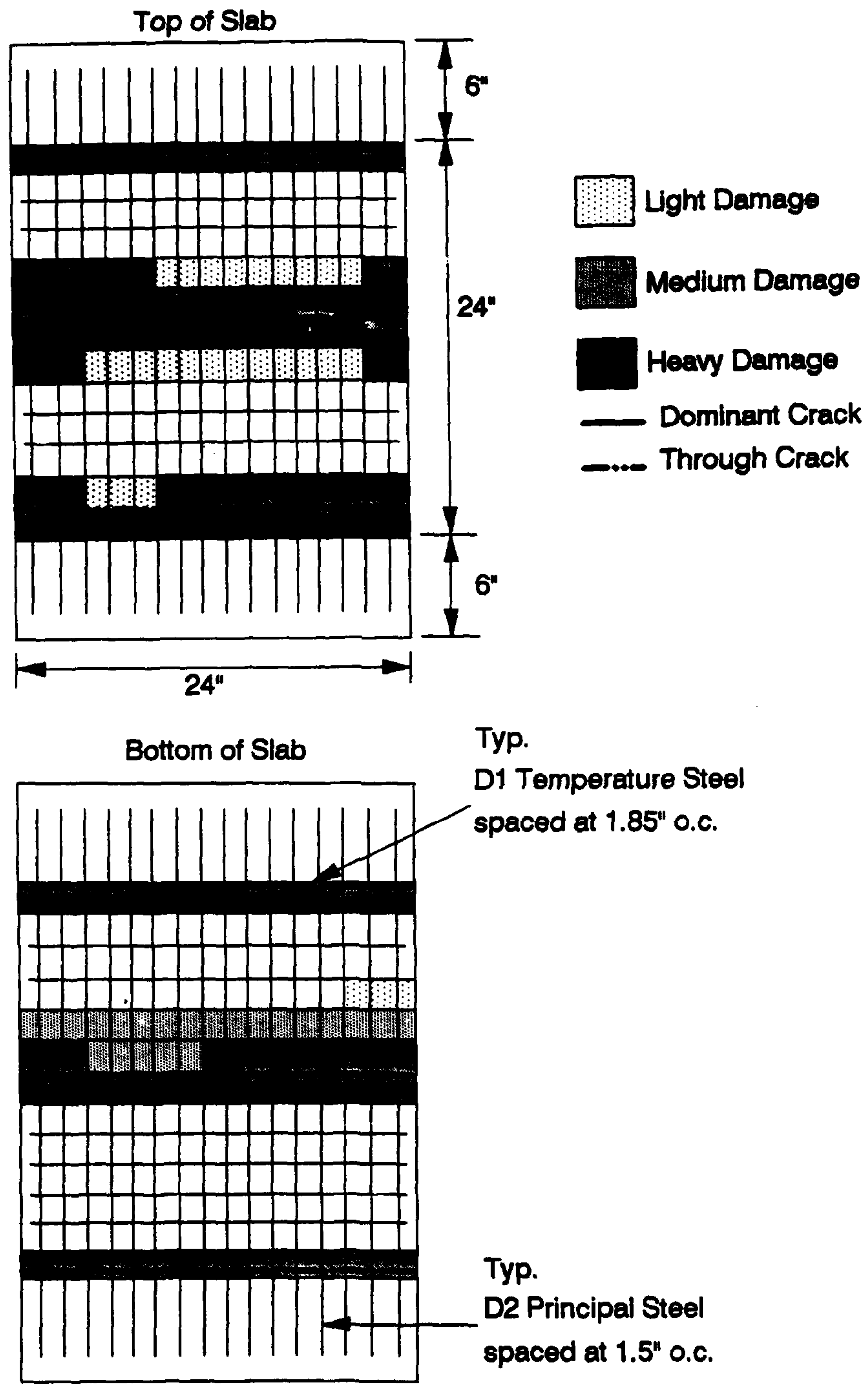


Figure 5.46. Damage Survey of Slab No. 12

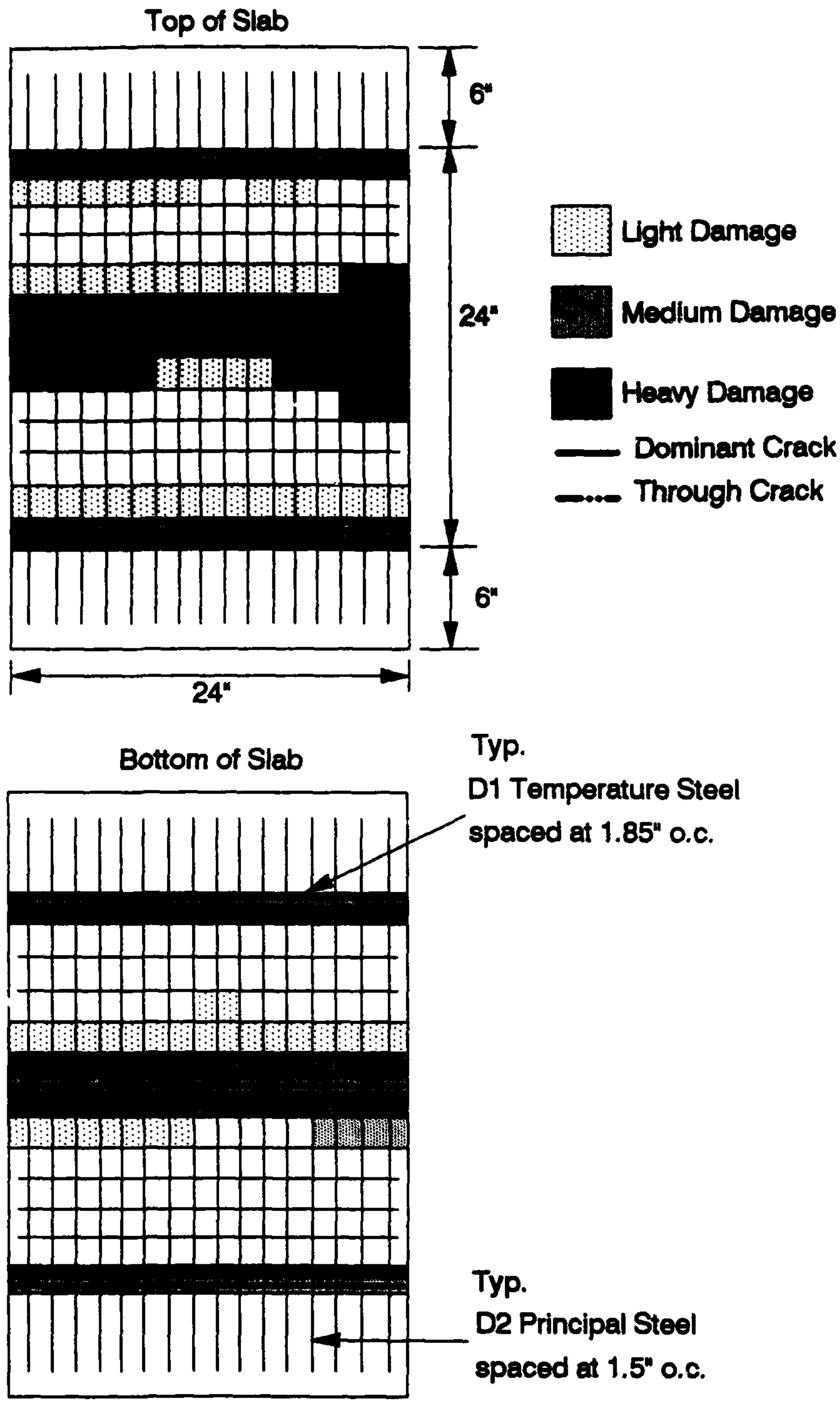


Figure 5.47. Damage Survey of Slab No. 13

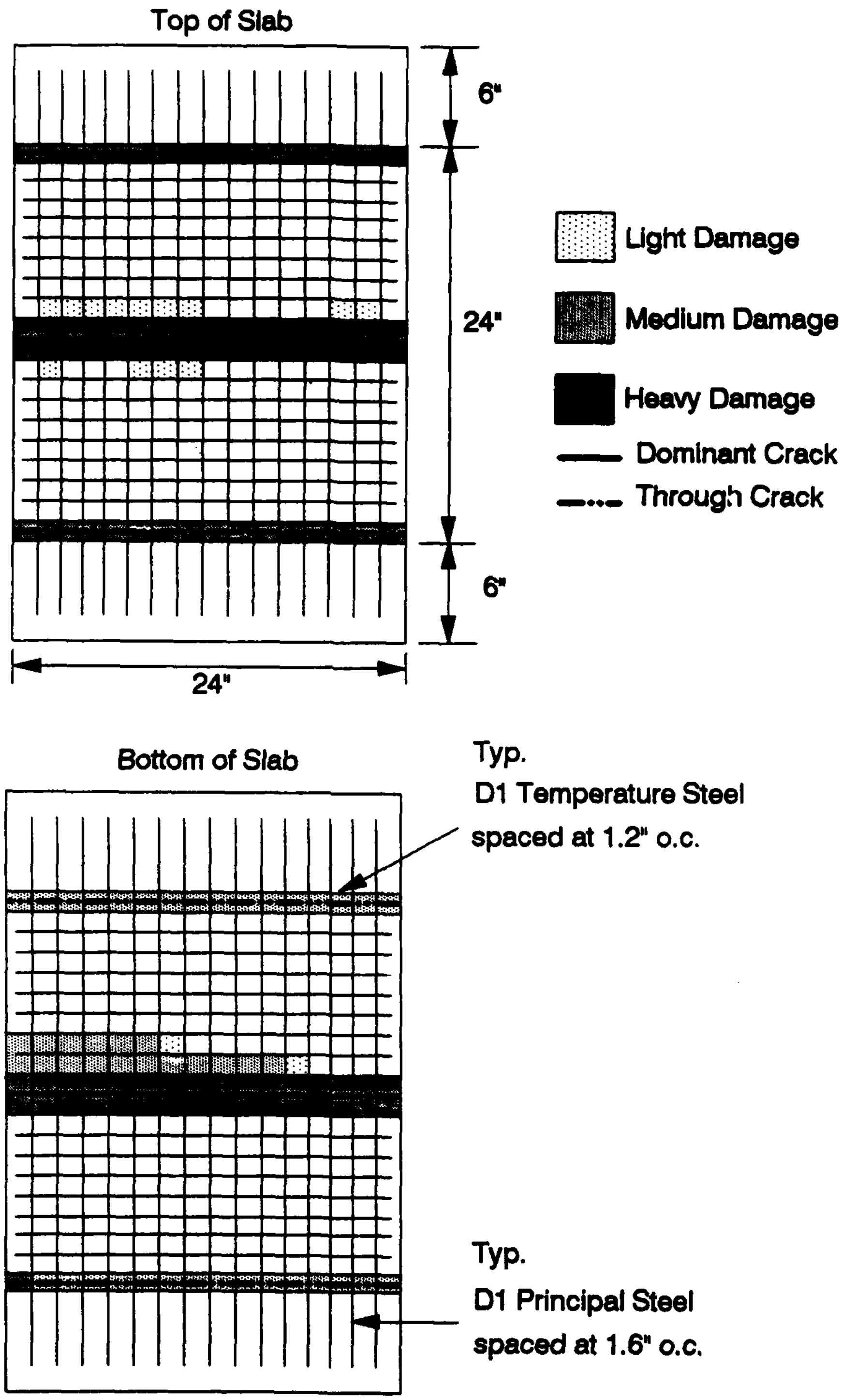


Figure 5.48. Damage Survey of Slab No. 14

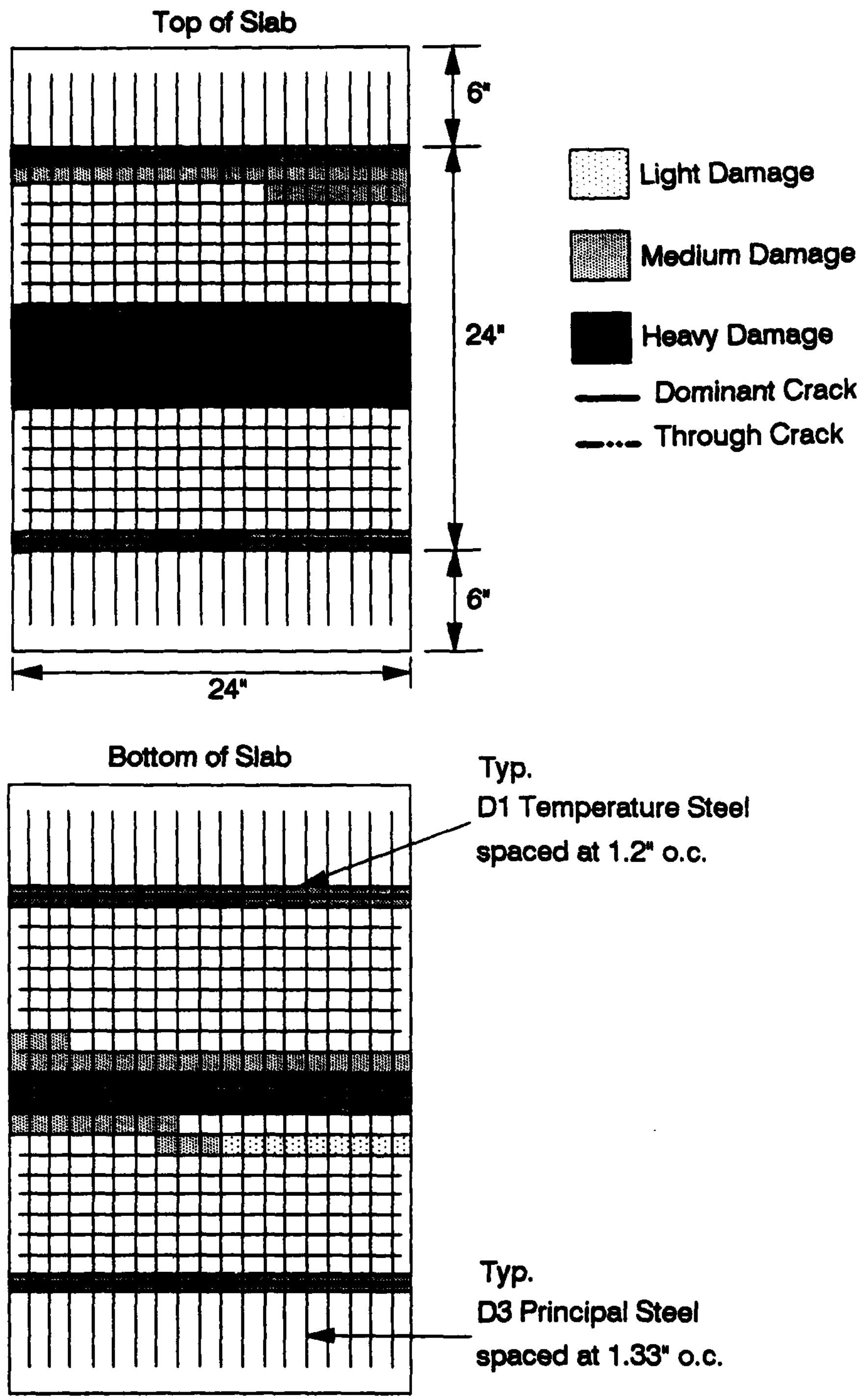


Figure 5.49. Damage Survey of Slab No. 15

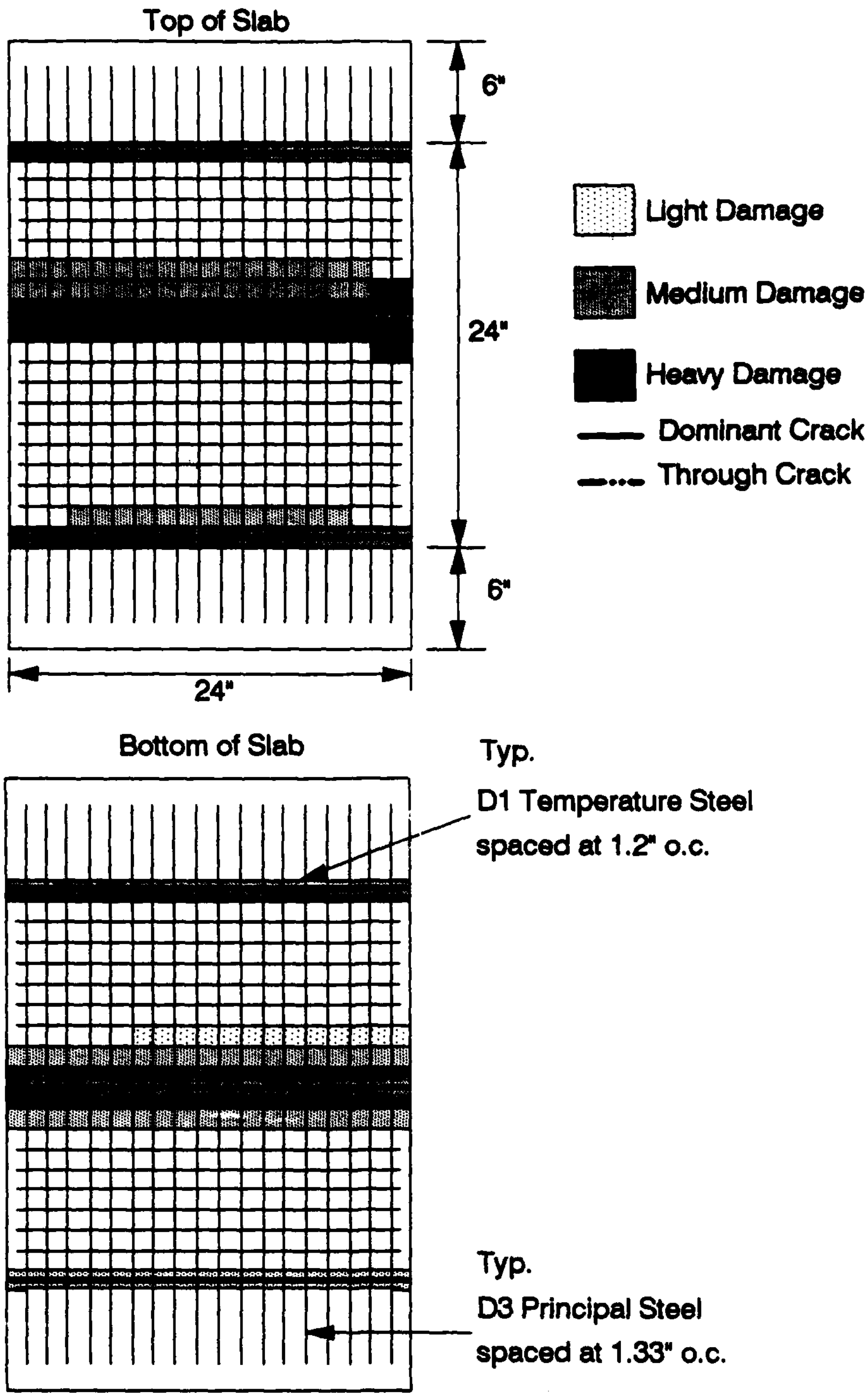


Figure 5.50. Damage Survey of Slab No. 16

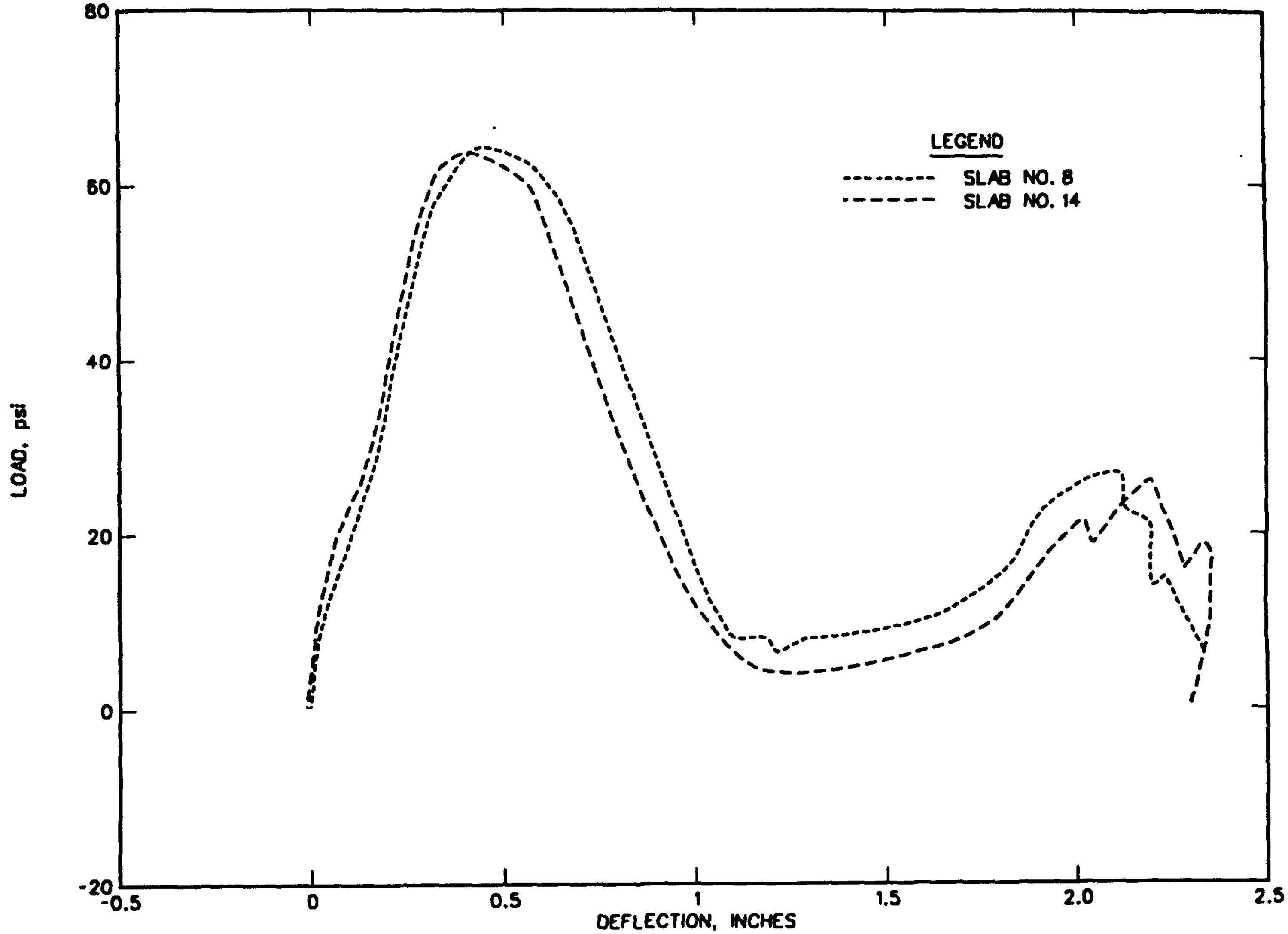


Figure 5.51. Smoothed Quarterspan Load-Deflection Curves for Slab Nos. 8 and 14

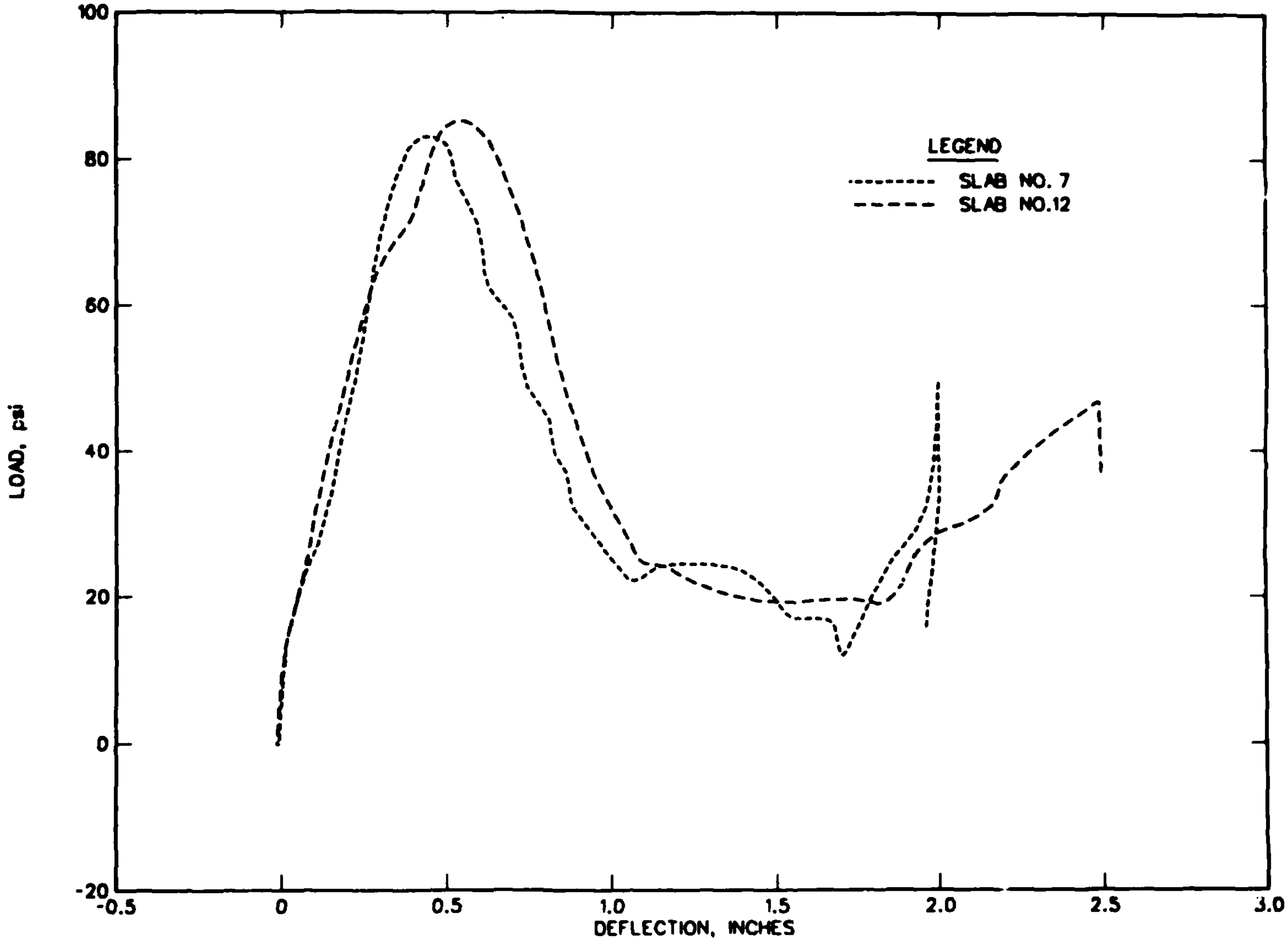


Figure 5.52. Smoothed Quarterspan Load-Deflection Curves for Slab Nos. 7 and 12

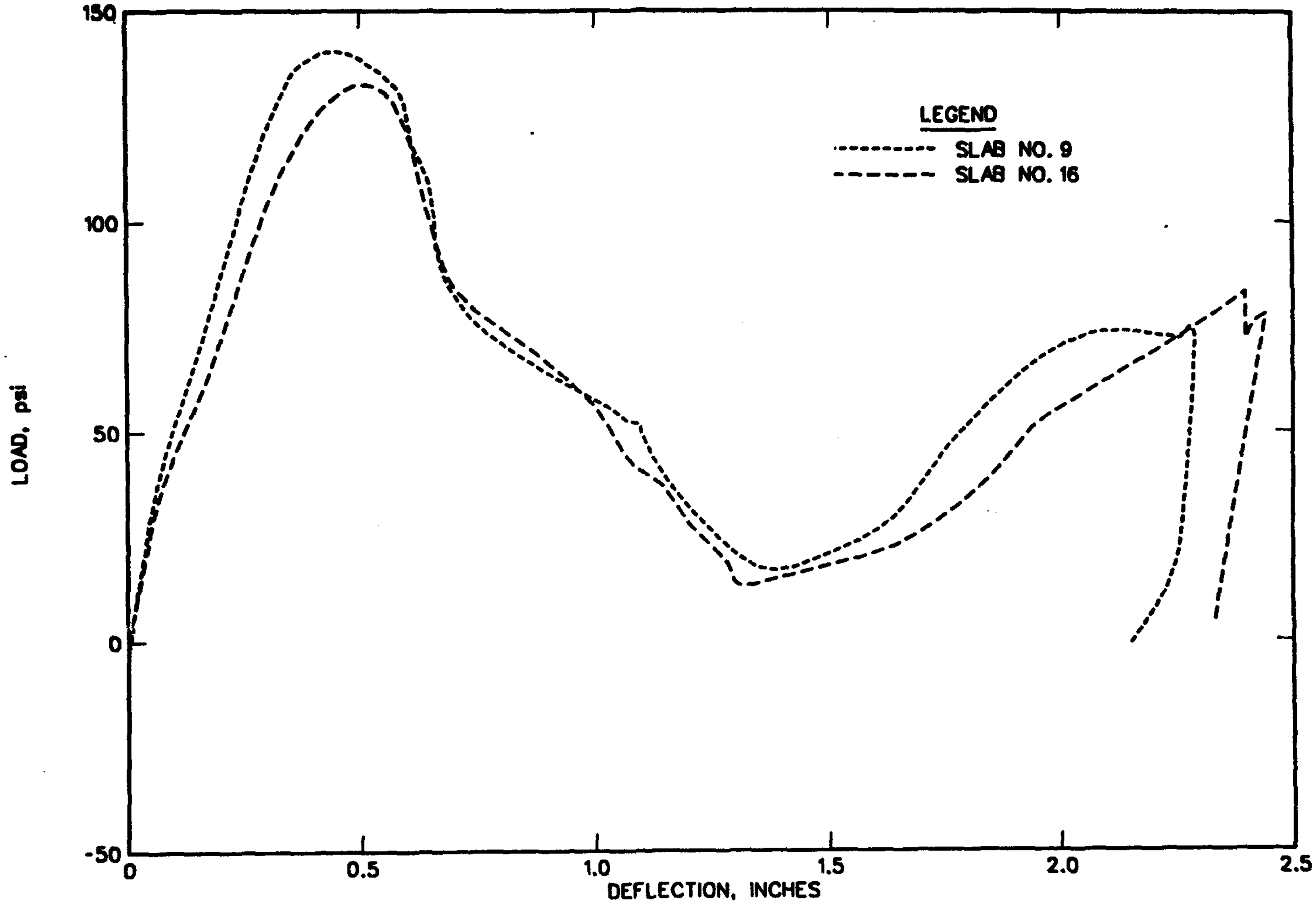
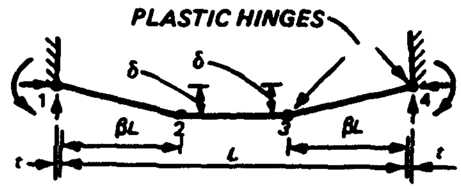
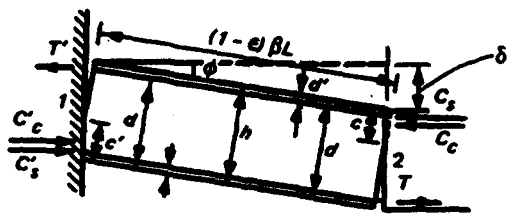


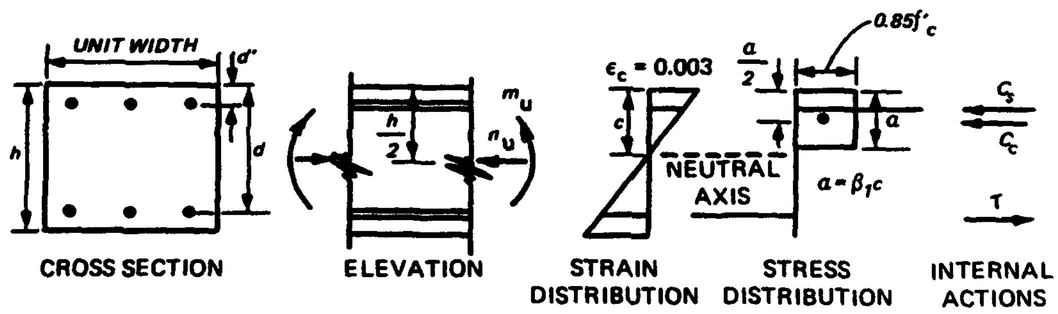
Figure 5.53. Smoothed Quarterspan Load-Deflection Curves for Slab Nos. 9 and 16



a. Geometry for Deformation of Restrained Strip



b. Portion of Strip Between Yield Sections



c. Assumed Conditions at Yield Section

Figure 5.54 Equilibrium and Deformations of a Slab Strip

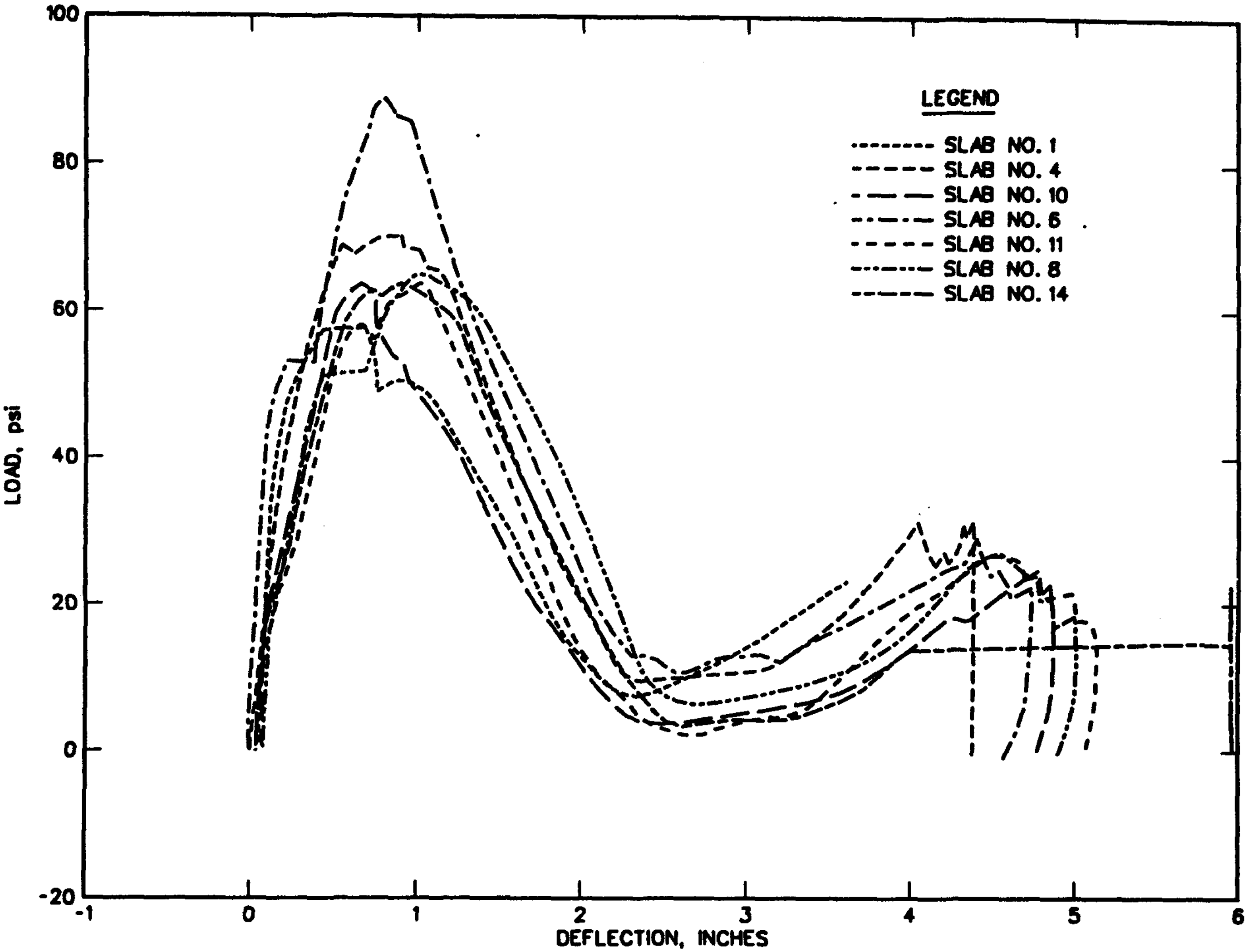


Figure 5.55. Composite of Smoothed Midspan Load Deflection Curves for Slabs with $\rho = 0.0025$

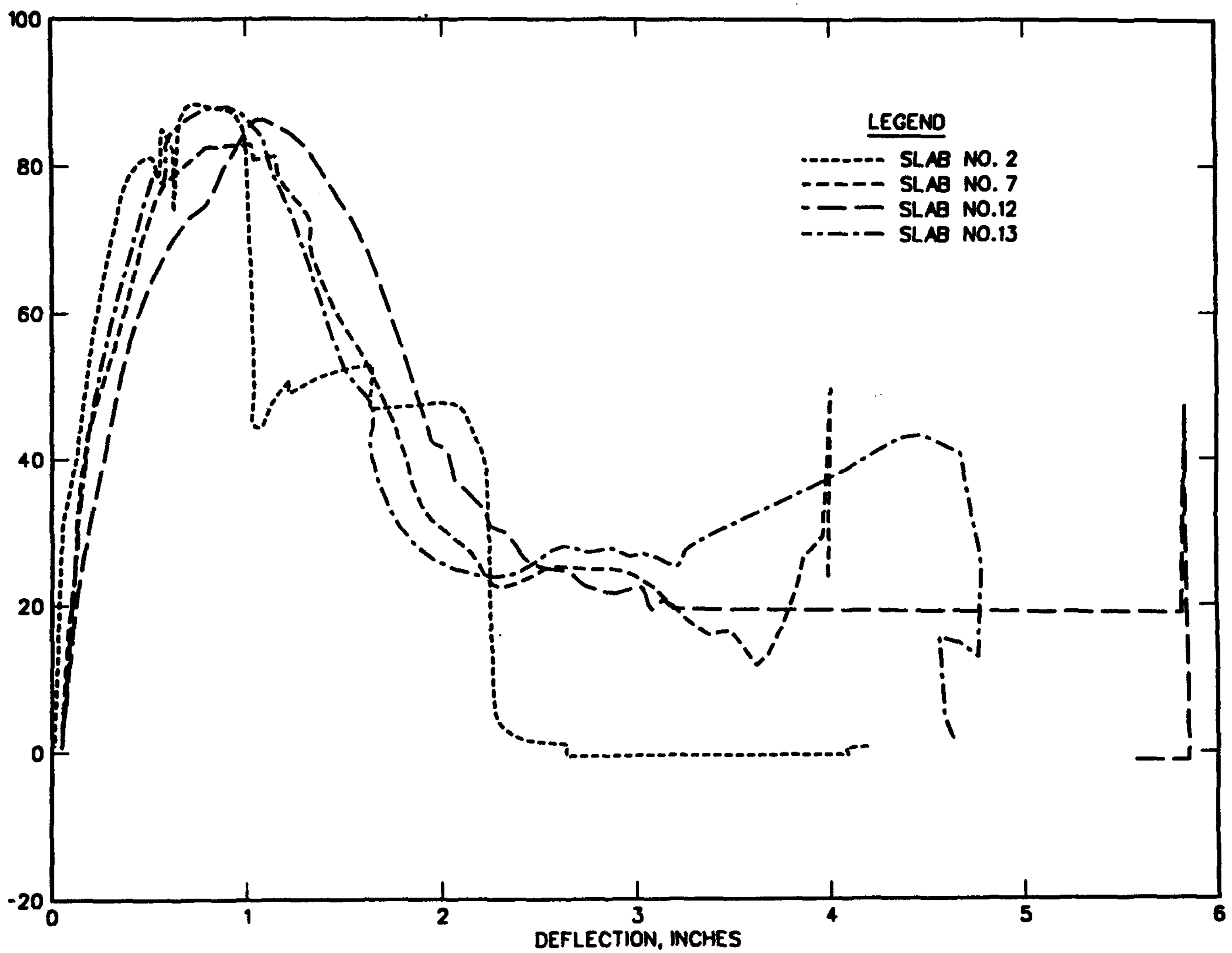


Figure 5.56. Composite of Smoothed Midspan Load Deflection Curves for Slabs with $\rho = 0.0056$

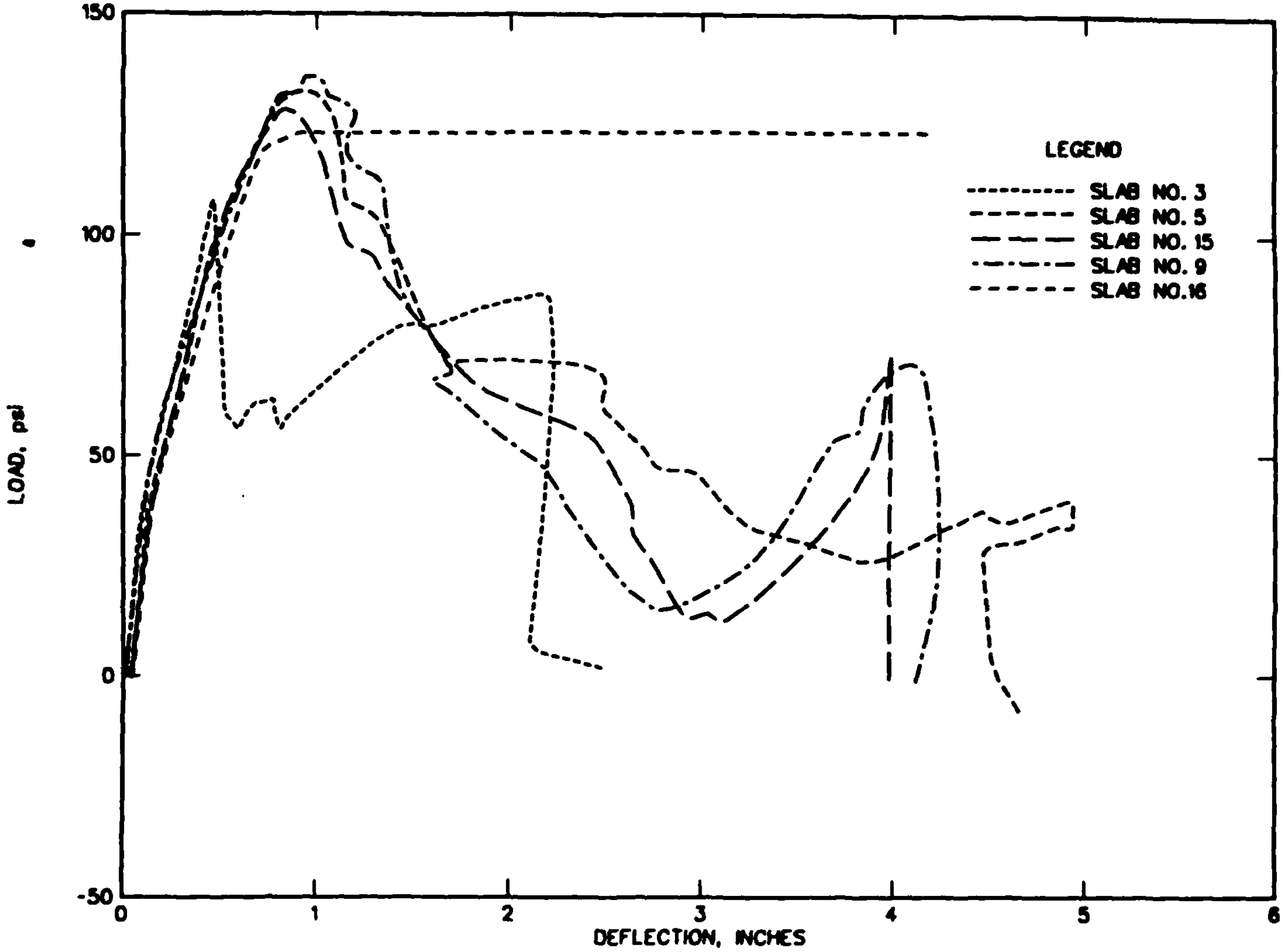


Figure 5.57. Composite of Smoothed Midspan Load Deflection Curves for Slabs with $\rho = 0.0097$

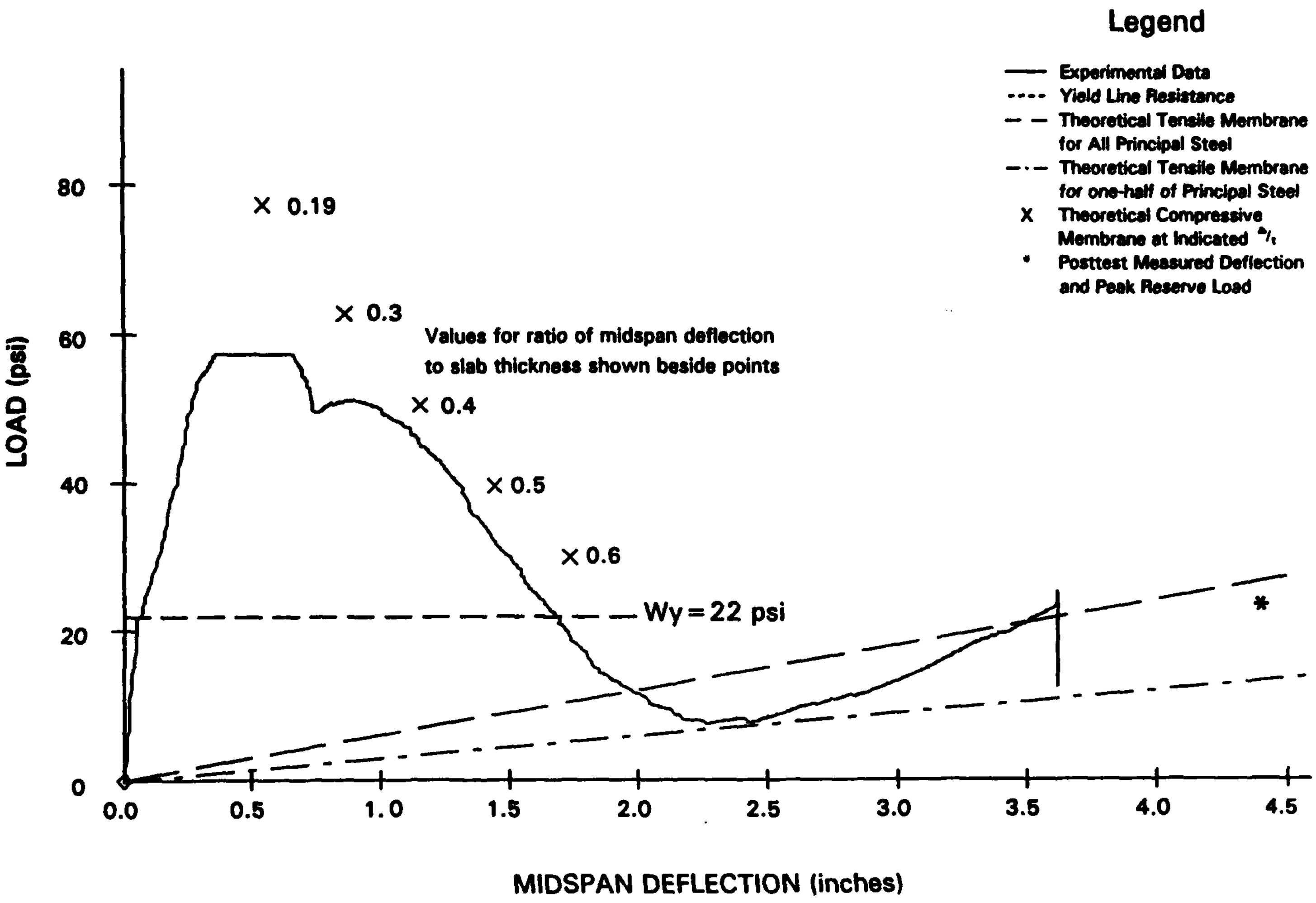


Figure 5.58. Experimental and Analytical Comparisons for Slab No. 1

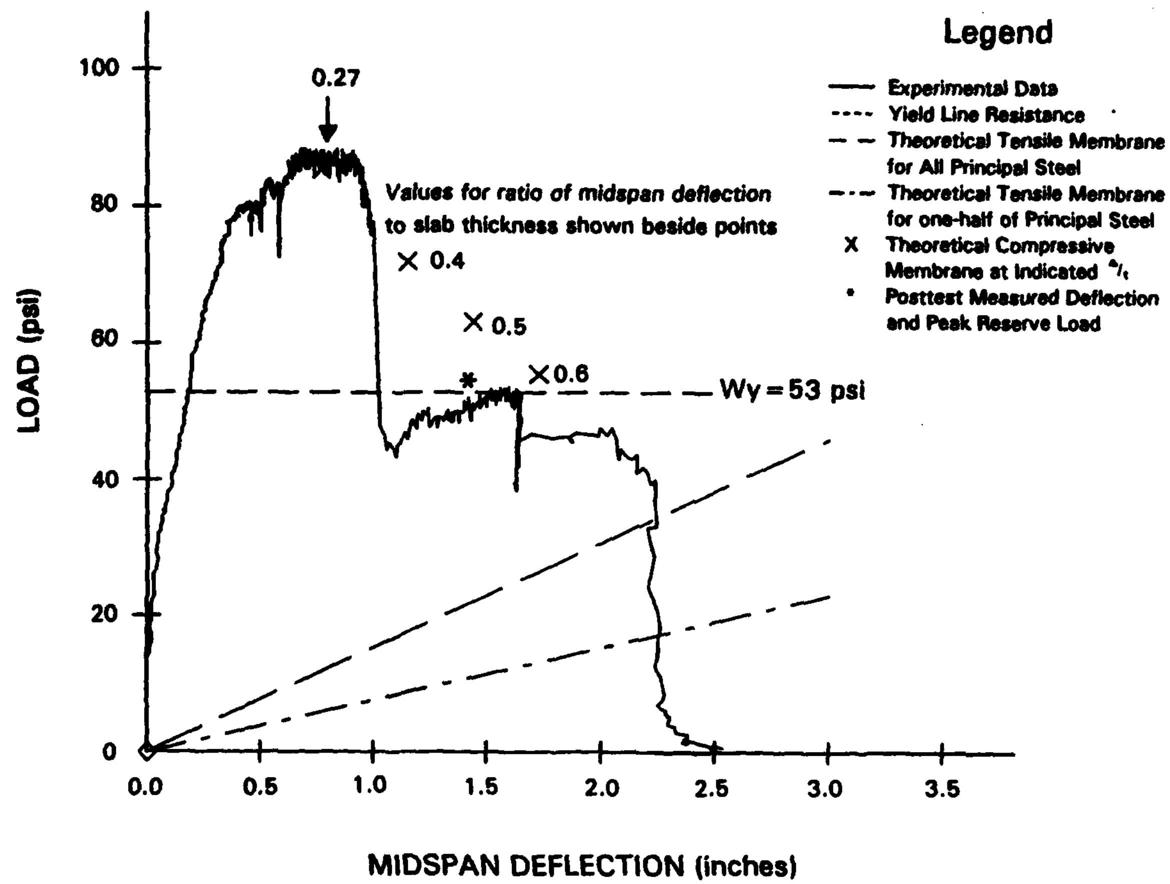


Figure 5.59. Experimental and Analytical Comparisons for Slab No. 2

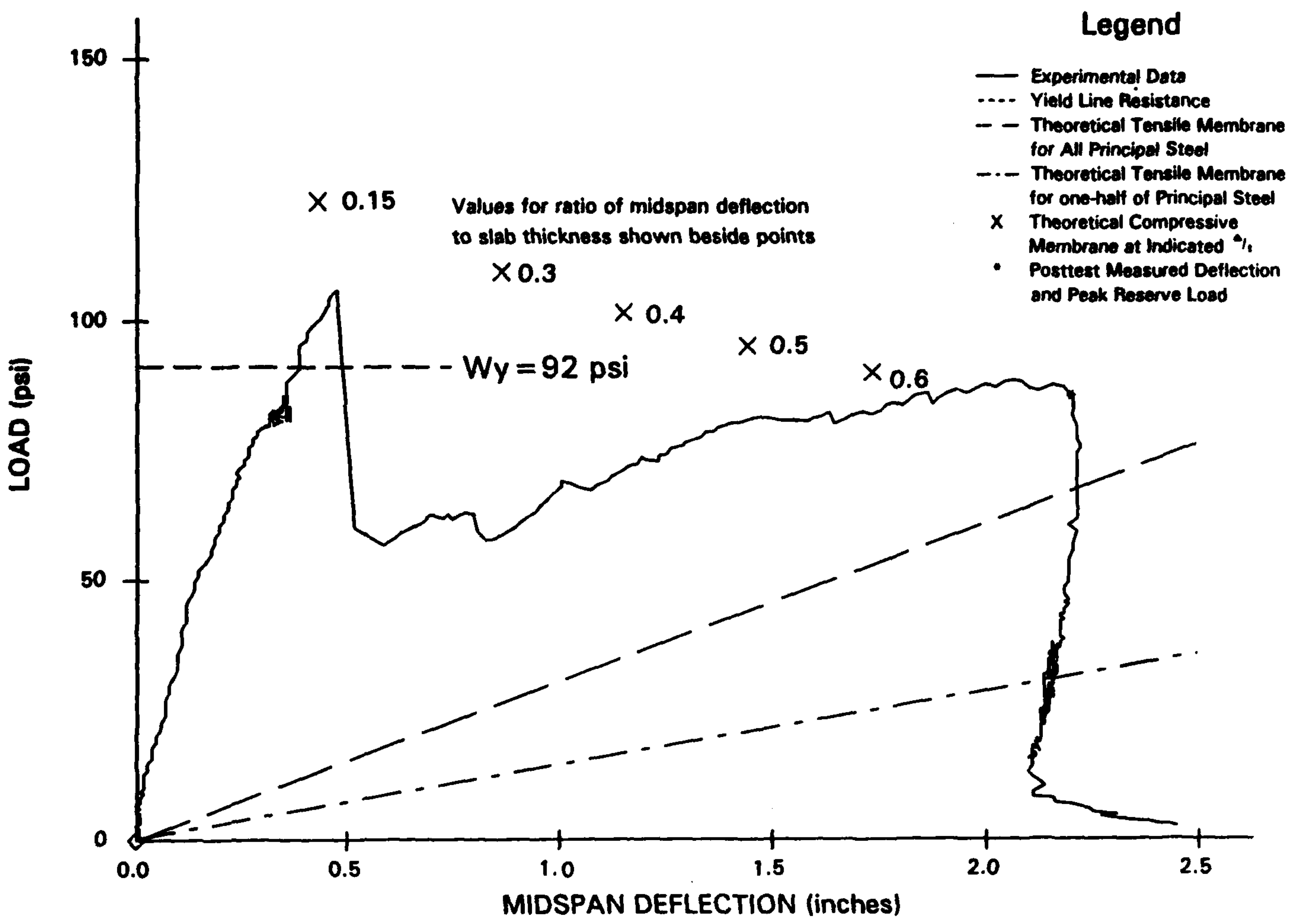


Figure 5.60. Experimental and Analytical Comparisons for Slab No. 3

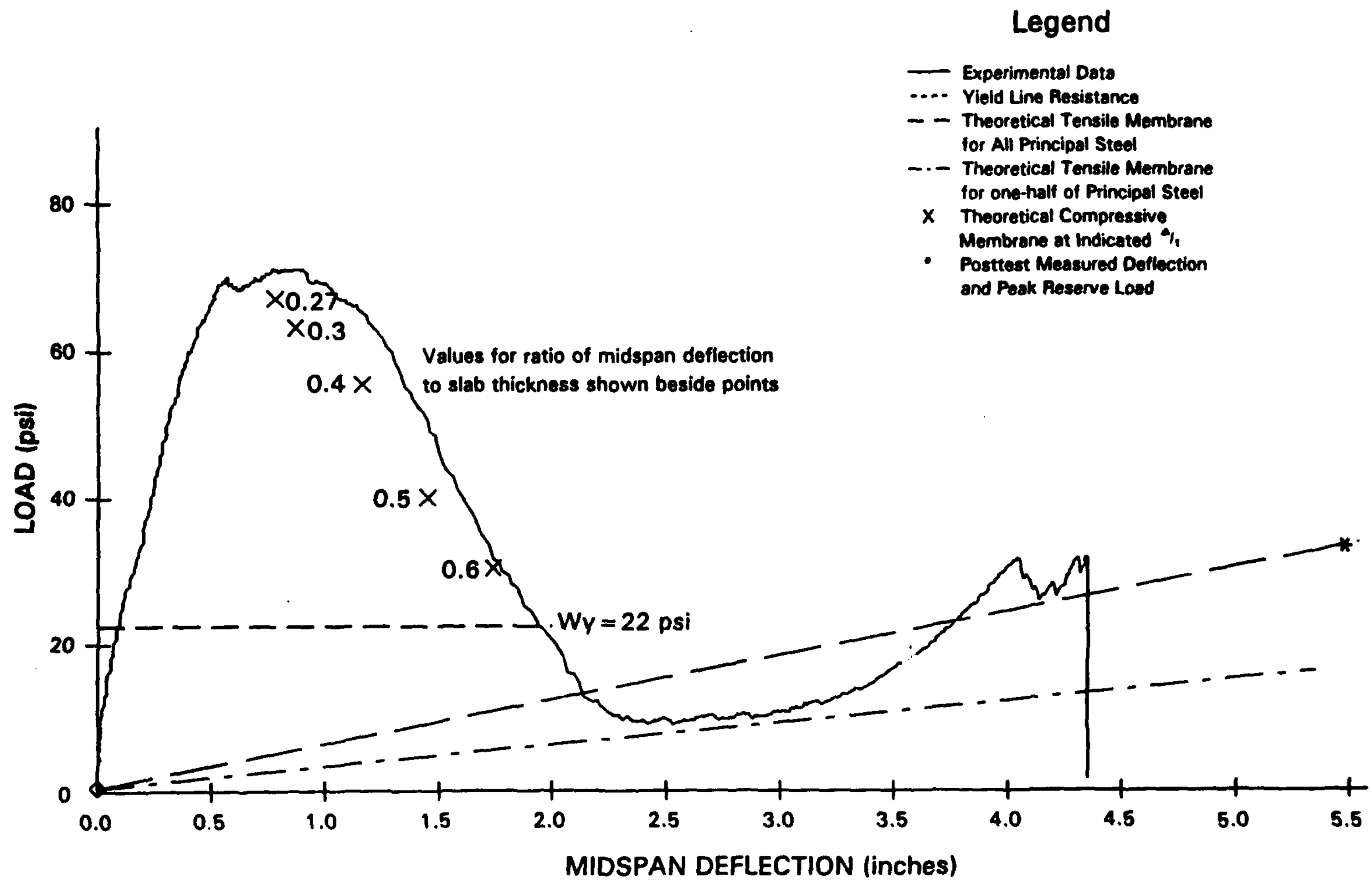


Figure 5.61. Experimental and Analytical Comparisons for Slab No. 4

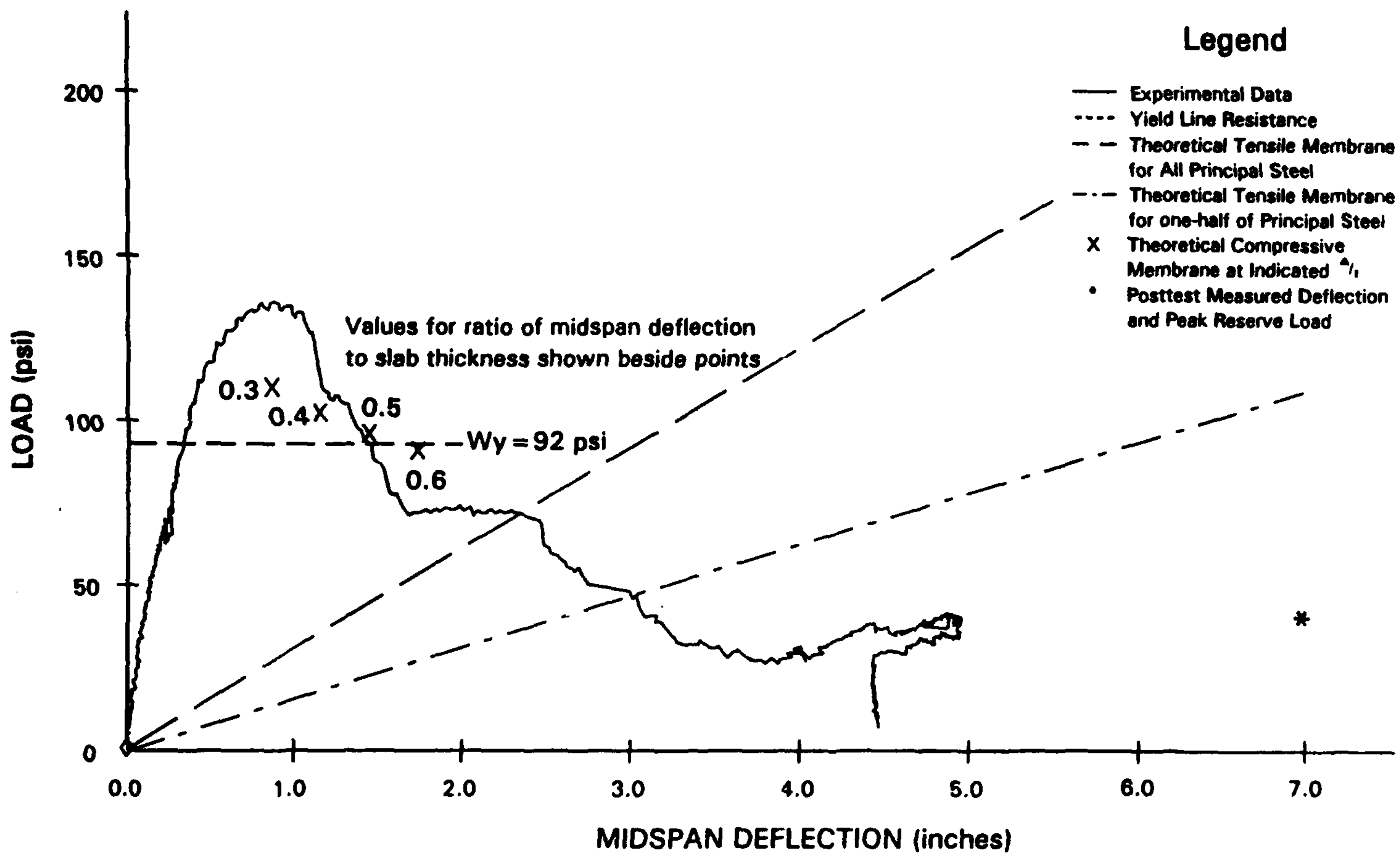


Figure 5.62. Experimental and Analytical Comparisons for Slab No. 5

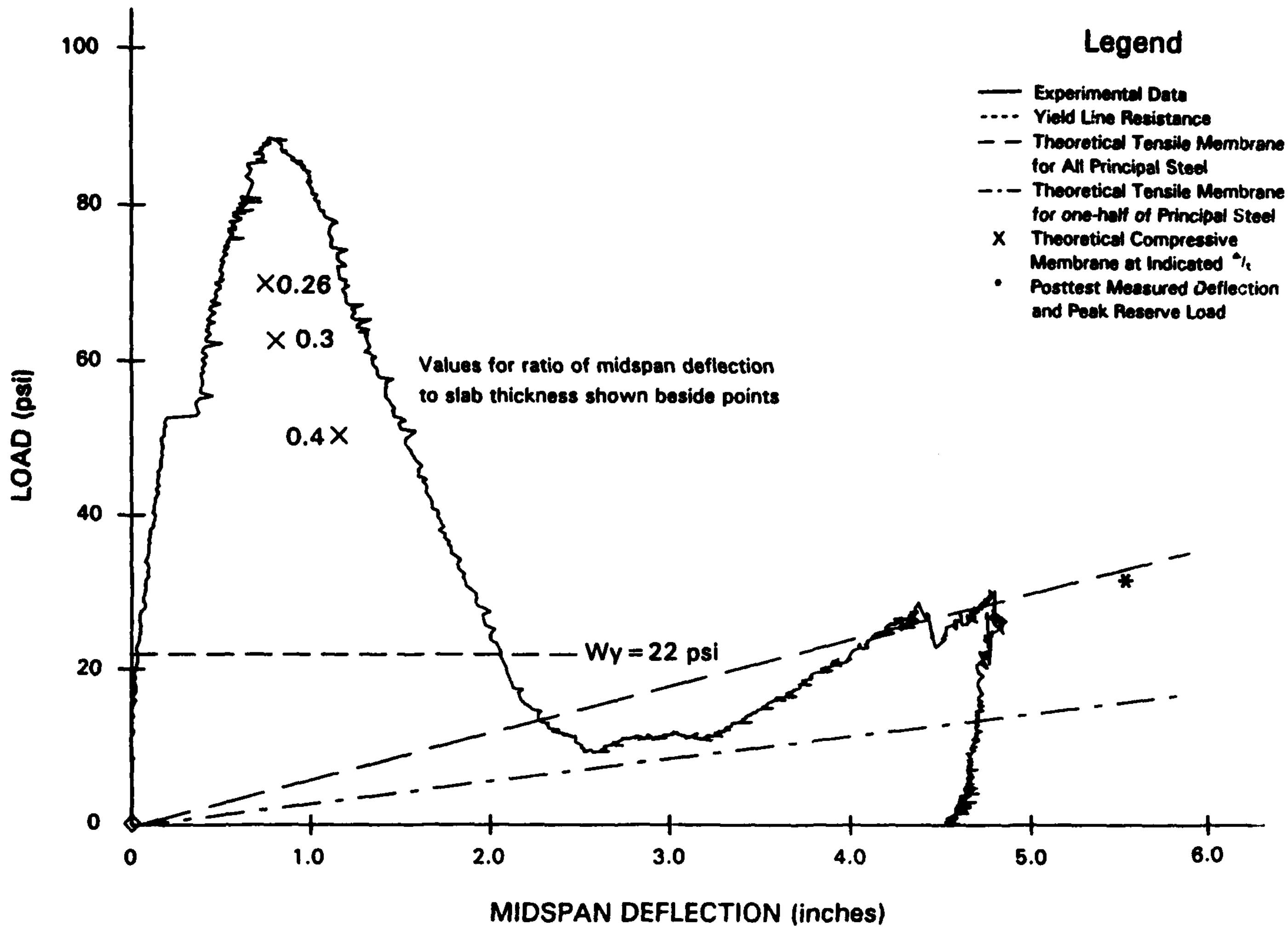


Figure 5.63. Experimental and Analytical Comparisons for Slab No. 6

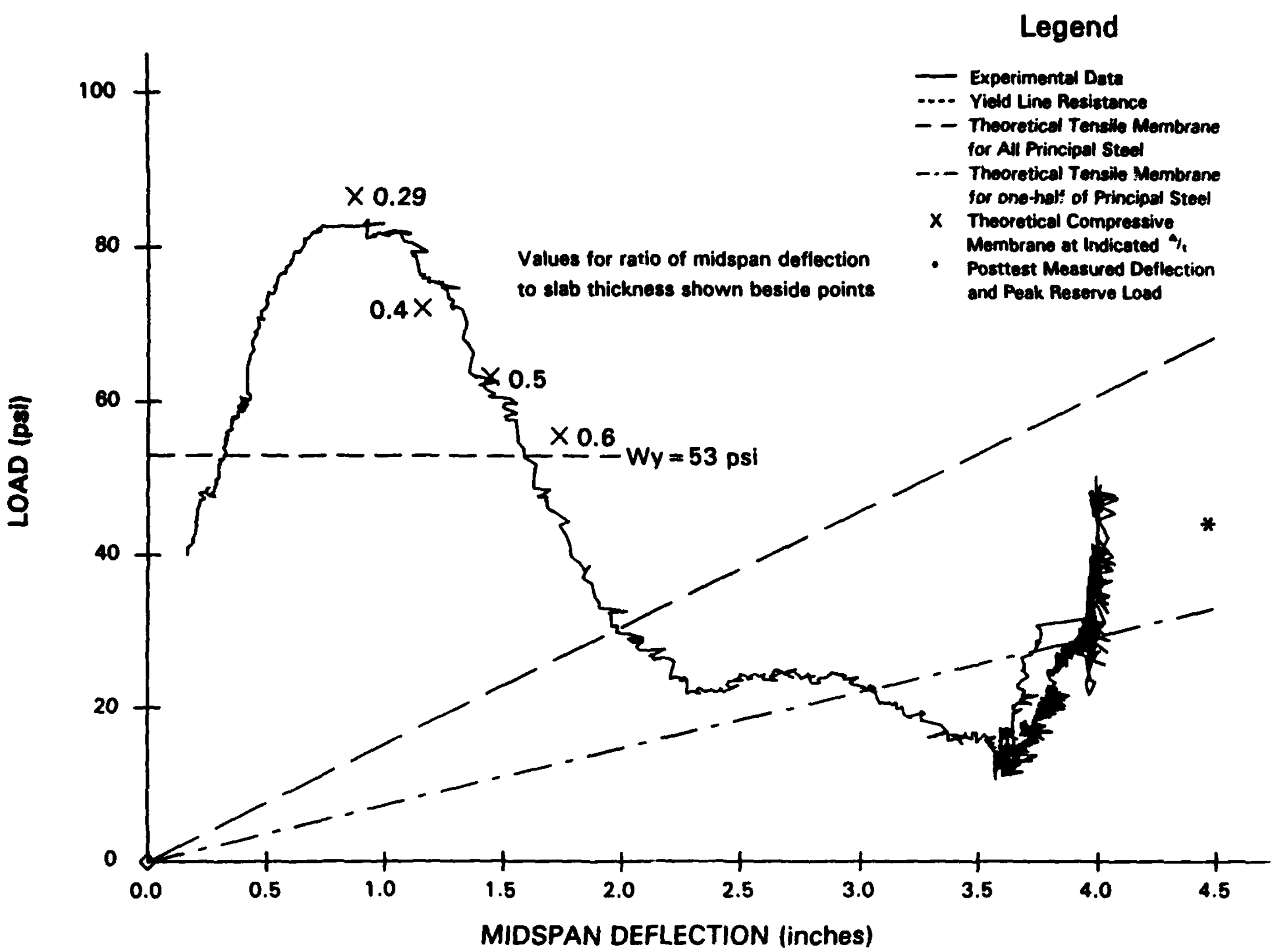


Figure 5.64. Experimental and Analytical Comparisons for Slab No. 7

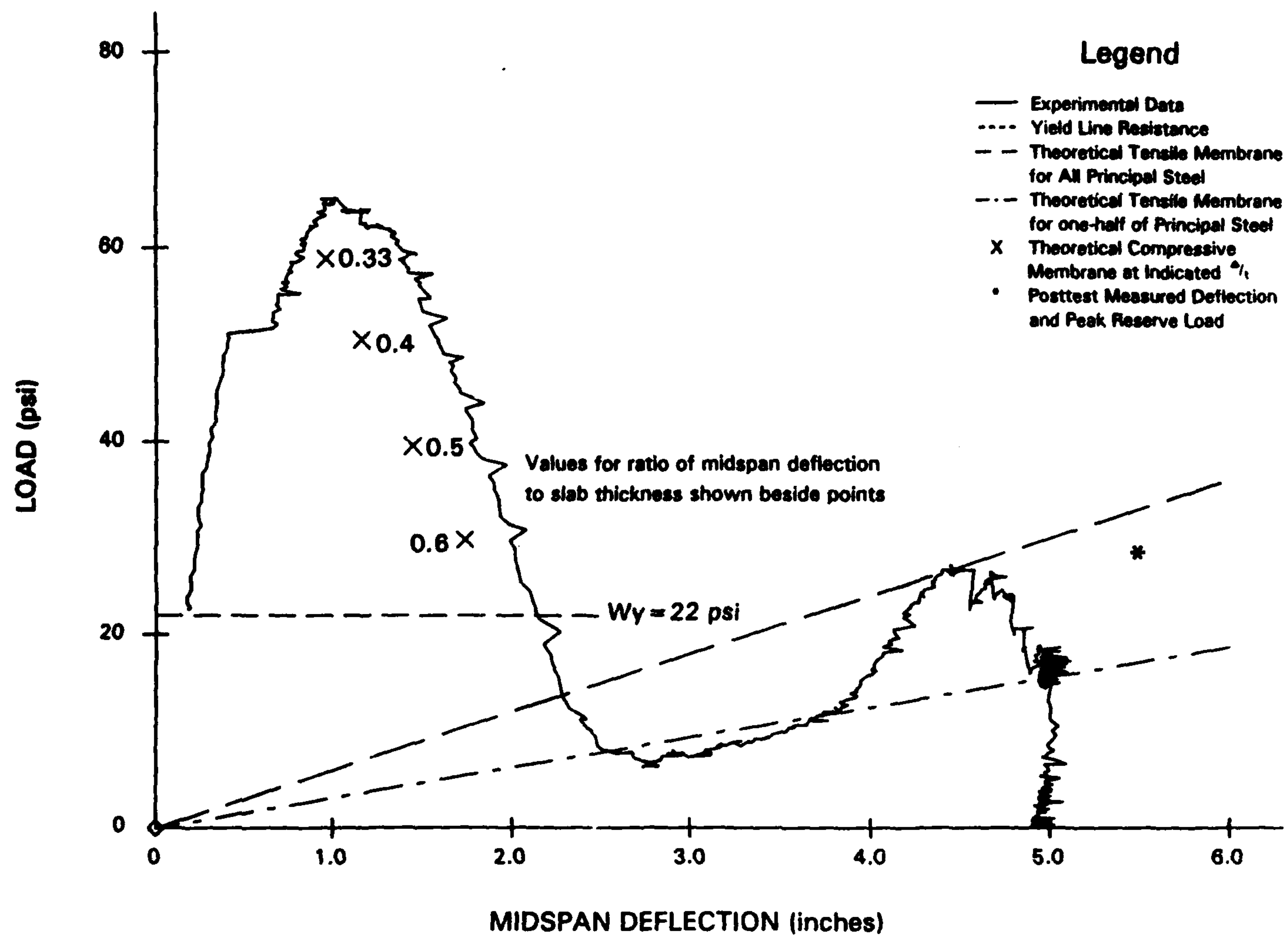


Figure 5.65. Experimental and Analytical Comparisons for Slab No. 8

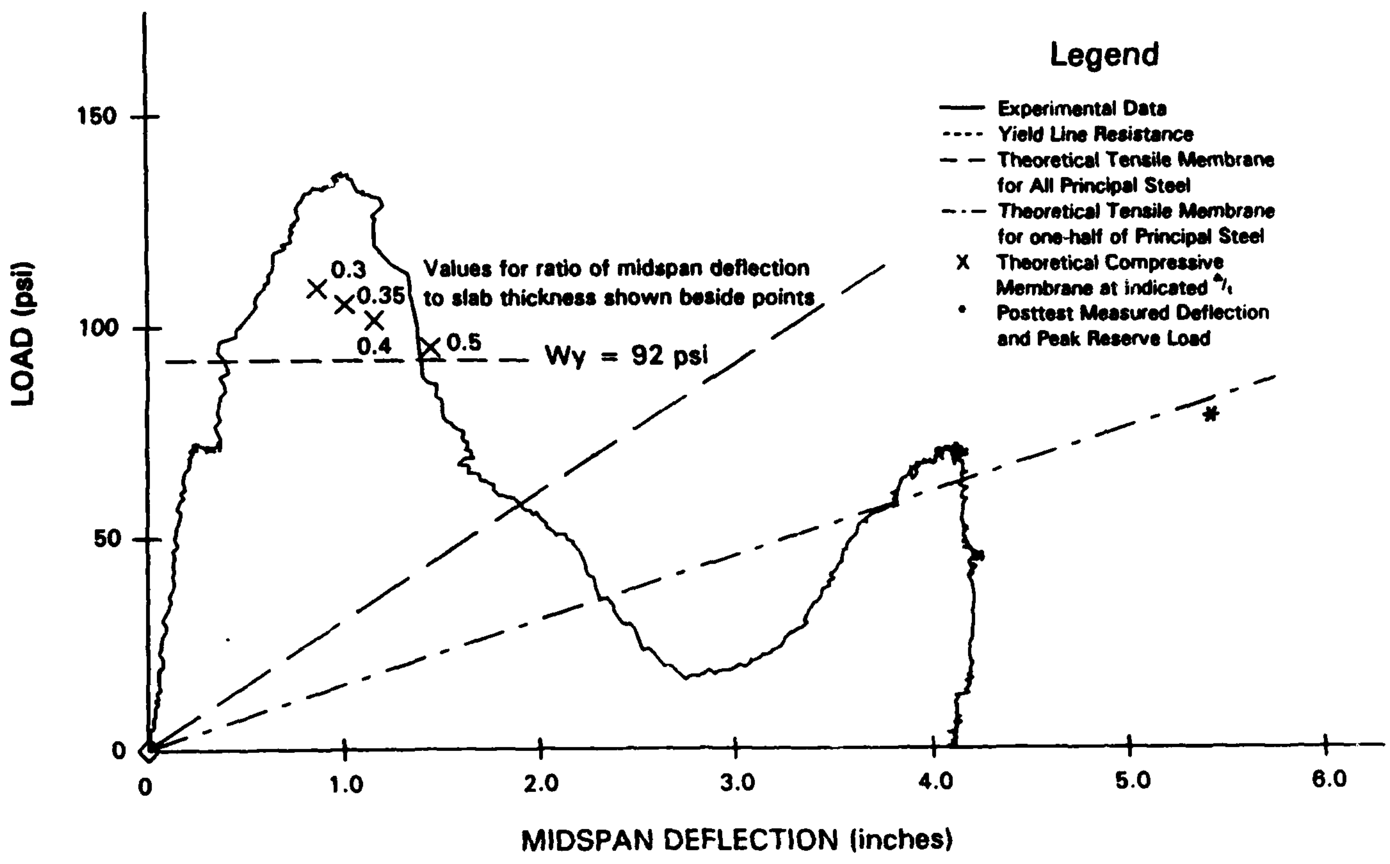


Figure 5.66. Experimental and Analytical Comparisons for Slab No. 9

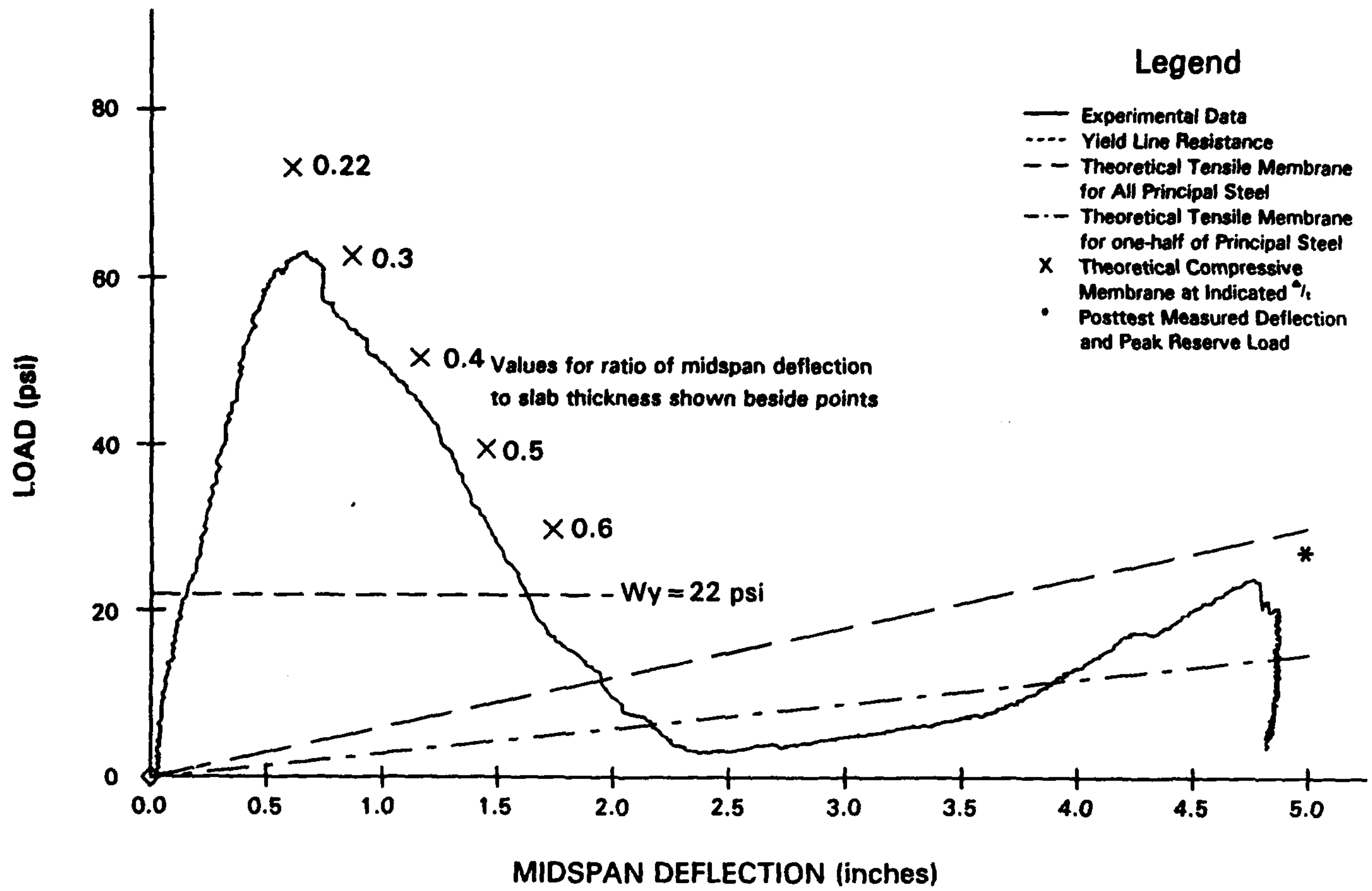


Figure 5.67. Experimental and Analytical Comparisons for Slab No. 10

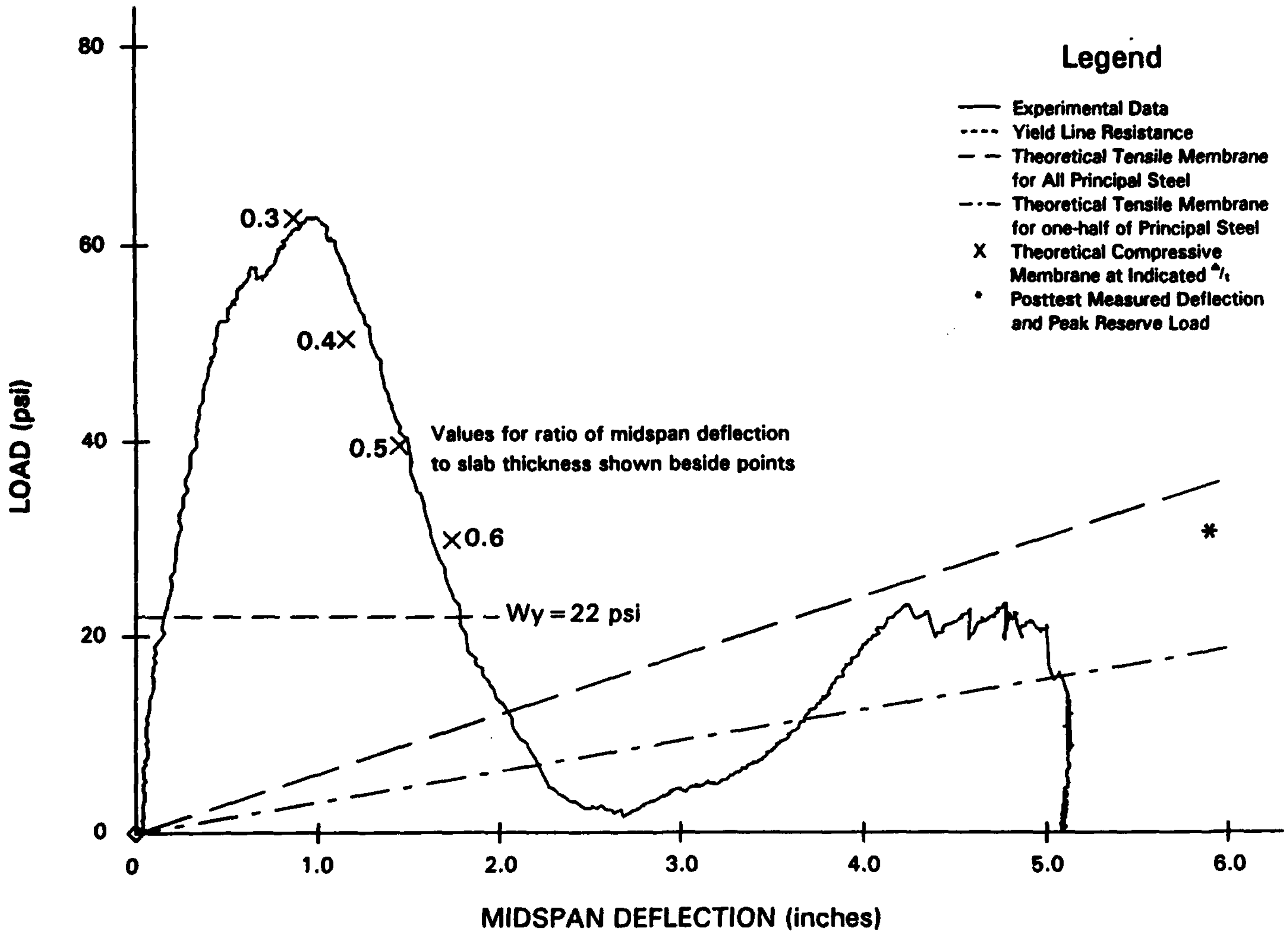


Figure 5.68. Experimental and Analytical Comparisons for Slab No. 11

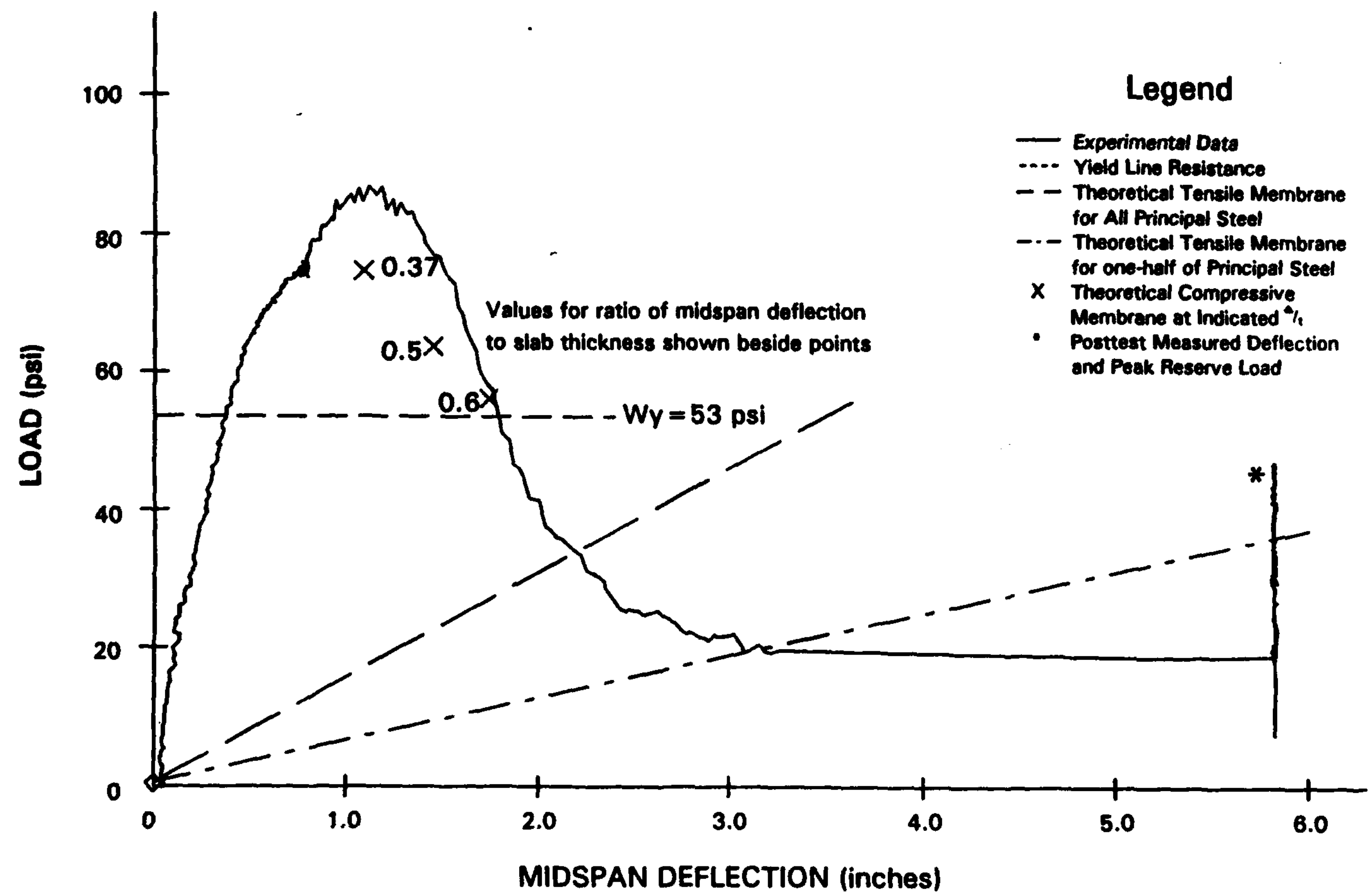


Figure 5.69. Experimental and Analytical Comparisons for Slab No. 12

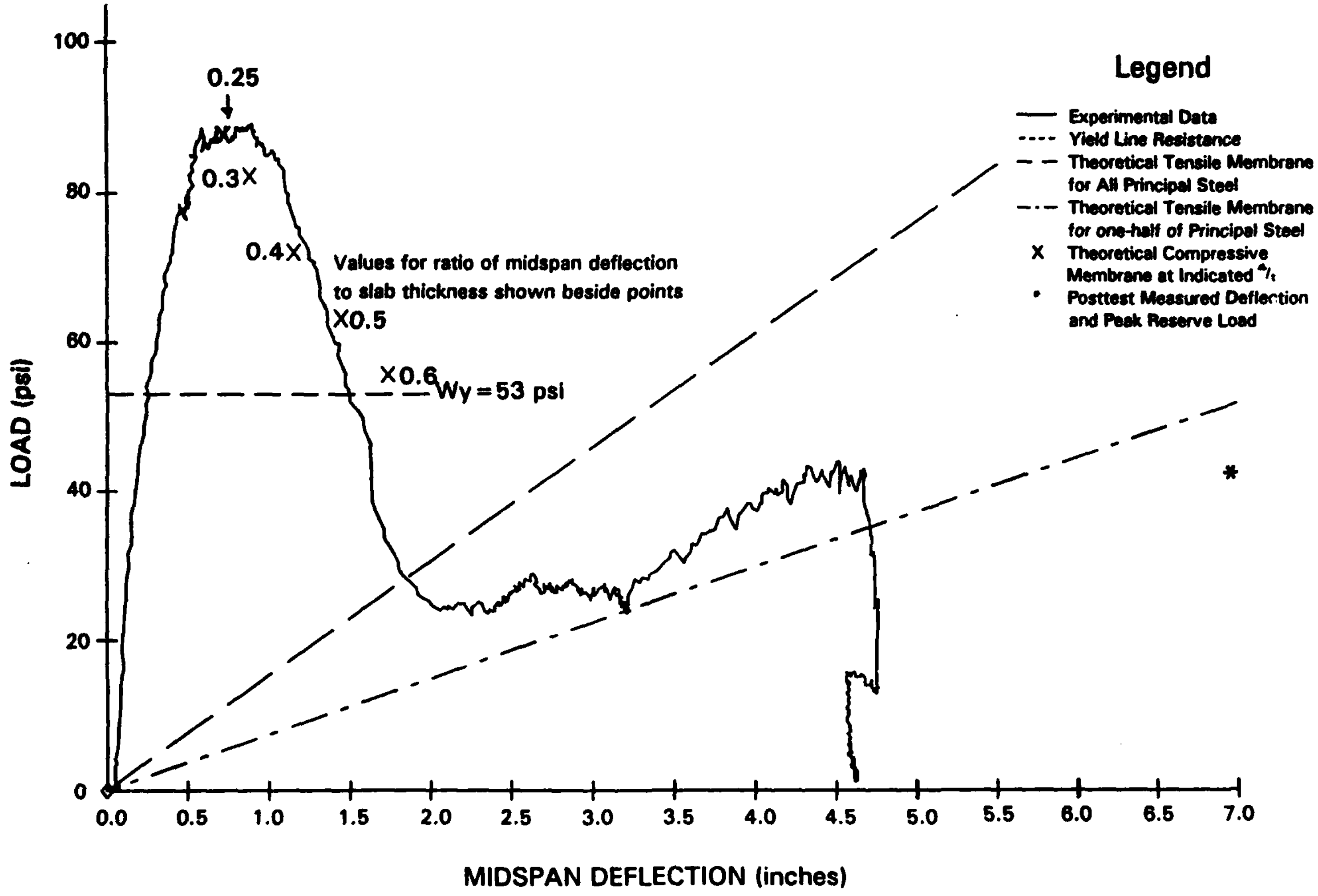


Figure 5.70. Experimental and Analytical Comparisons for Slab No. 13

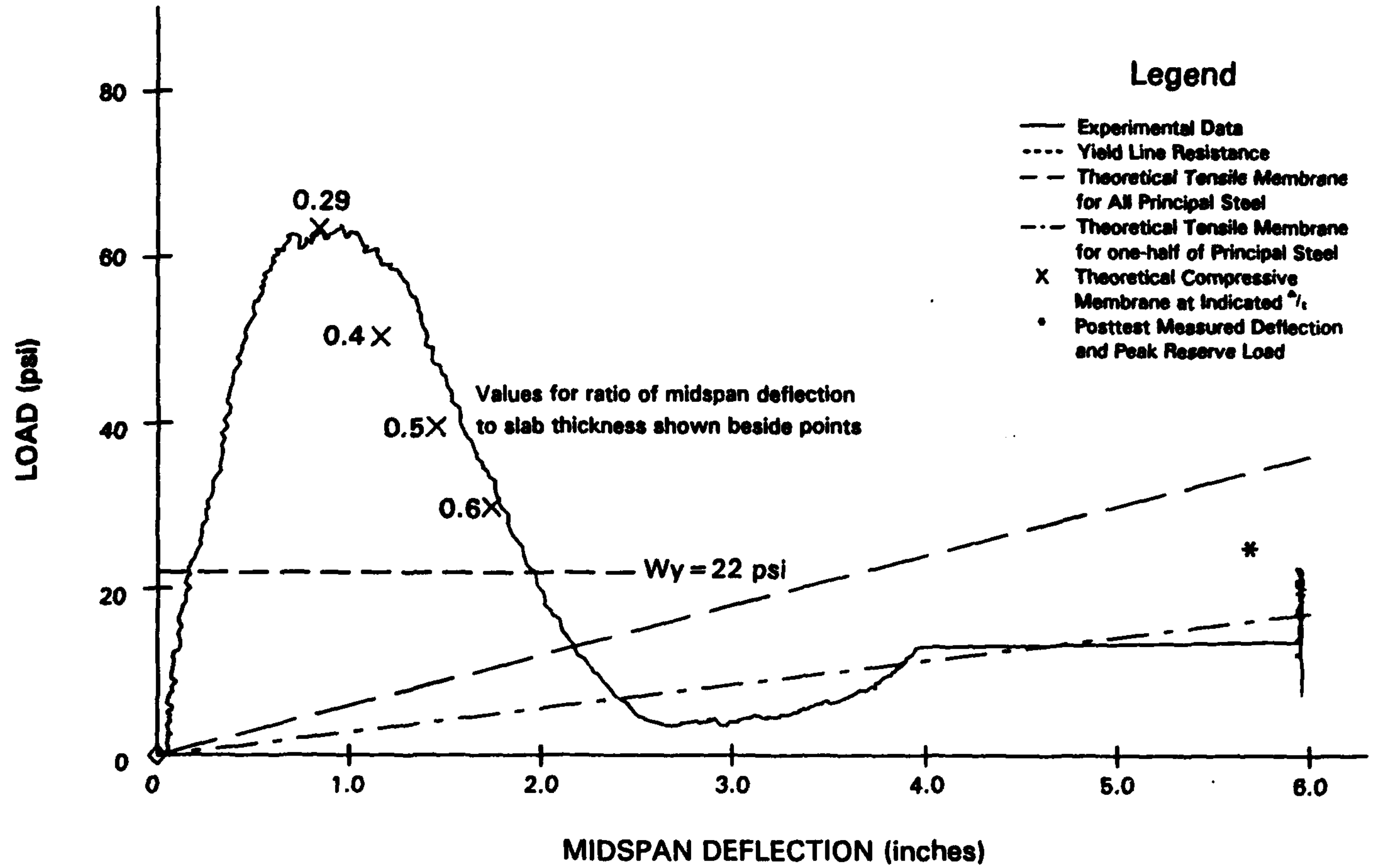


Figure 5.71. Experimental and Analytical Comparisons for Slab No. 14

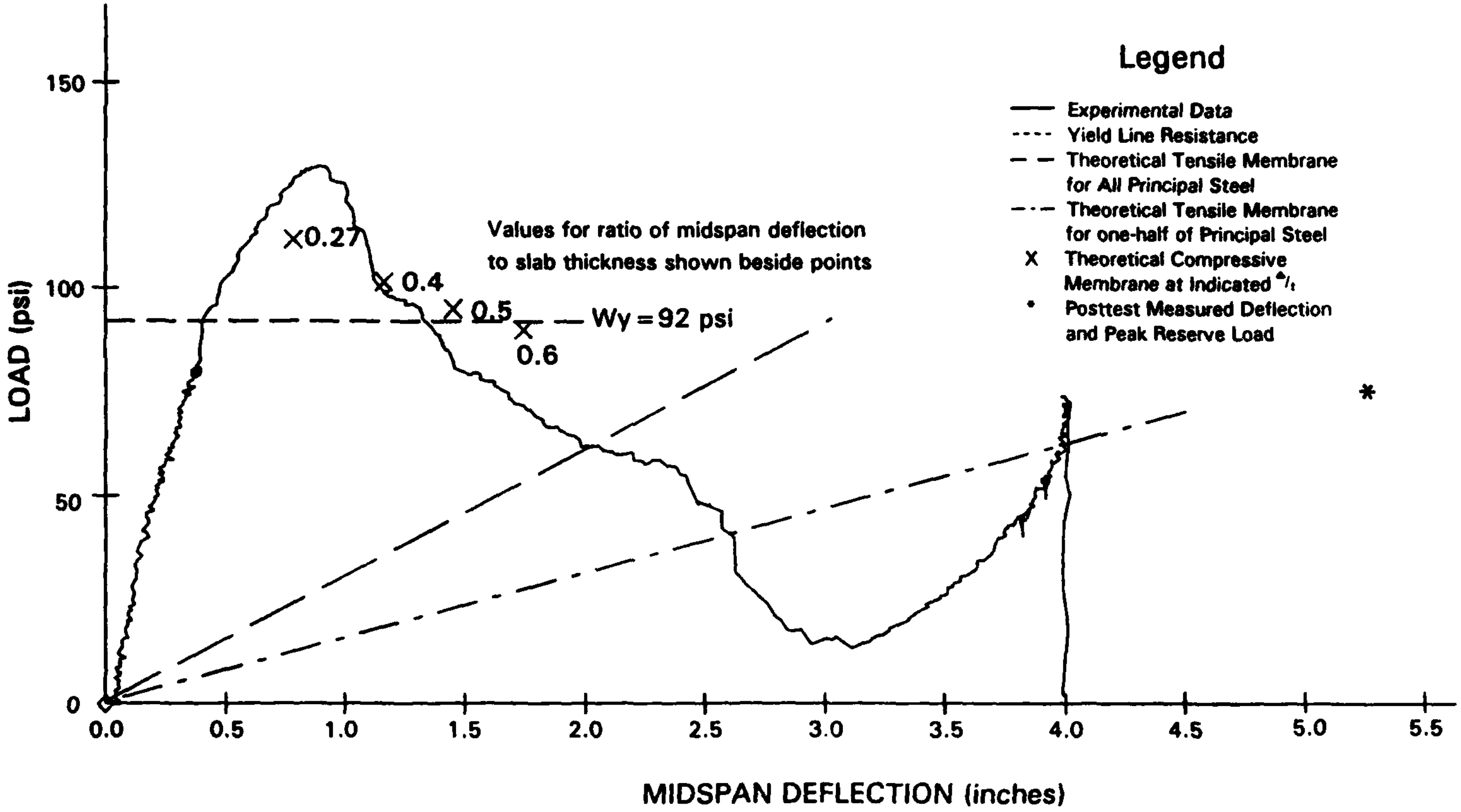


Figure 5.72. Experimental and Analytical Comparisons for Slab No. 15

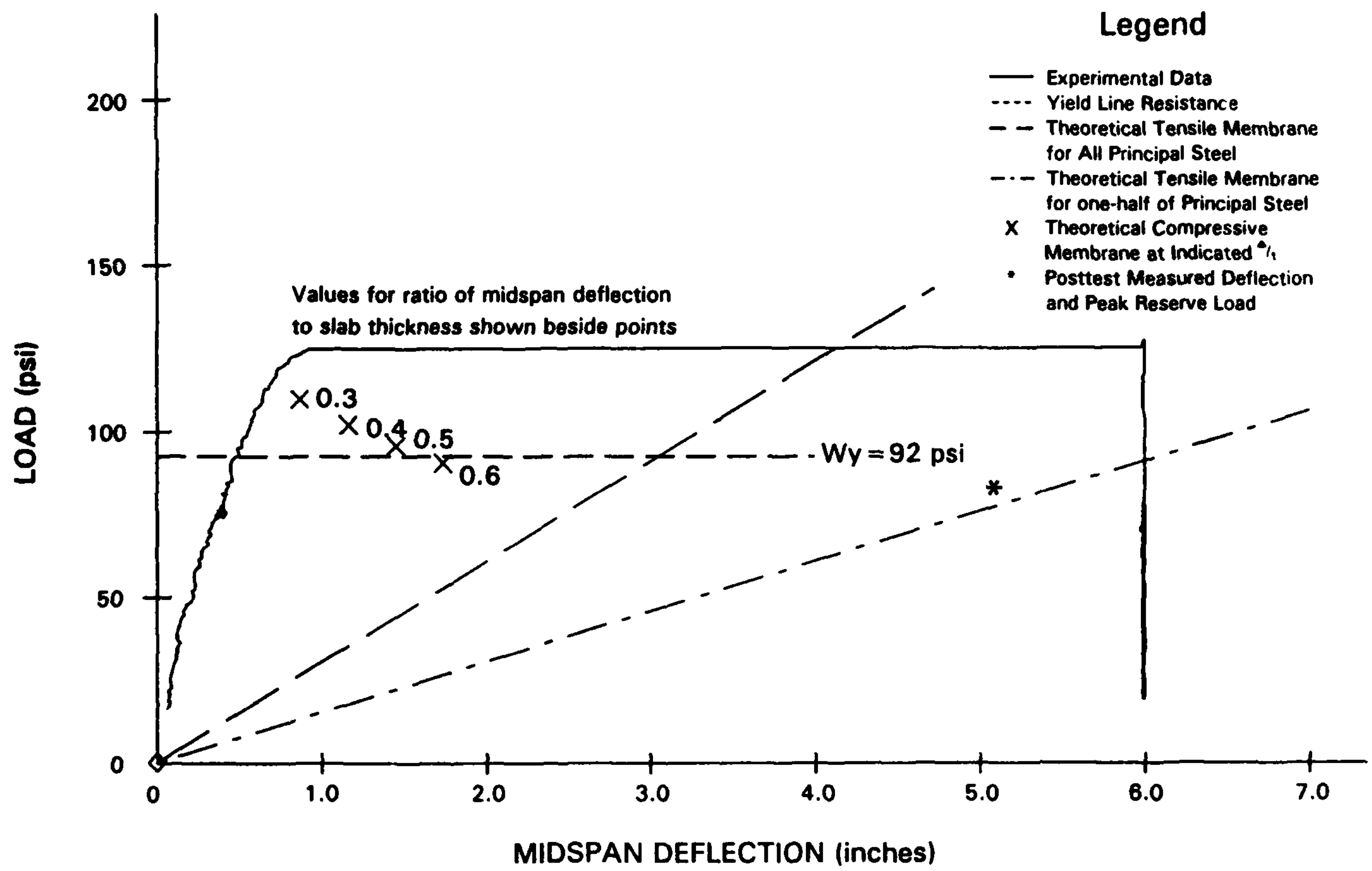


Figure 5.73. Experimental and Analytical Comparisons for Slab No. 16

CHAPTER 6

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 Summary

In addition to the presentation and analysis of the data produced during this investigation, significant contributions of this thesis included the collection and evaluation of existing experimental data. The review of existing data indicated that design criteria for protective or blast-resistant structures are overly conservative in shear reinforcement requirements and associated response limits. The restricted use of stirrups and the low levels of attainable response (support rotations) assigned by TM 5-1300 to reinforced concrete slabs are based on incomplete experimental studies. No valid comparisons of the effects of stirrups and lacing bars on slab response exist in the data base; however, various experiments with various objectives were conducted on reinforced concrete slabs during the late 1970's until the present. This author's awareness of the various experiments prompted the data review presented in this thesis and the development of ETL 1110-9-7. The ETL was developed during the early phase (data review) of this study; therefore, it was attempt to use existing data to reduce the conservatism found in design manuals.

The experimental investigation of this study was a first step toward a thorough comparison and evaluation of the effects of stirrups and lacing bars on the large-deflection behavior of the slabs. In general, there were no significant differences in the behavior of the slabs with lacing bars and the slabs with stirrups

that were experimentally evaluated in this study. Care must be taken when extending the results of the static experiments of this study to the design of slabs in a dynamic environment. One should not expect the failure modes of the slabs in this study to replicate that of slabs subjected to blast from very close-in detonations (perhaps, those at scaled ranges less than about 1.0 ft/lb$^{1/3}$); yet, the results are appropriate for slabs subjected to blast from far-away detonations or when the loading is composed primarily of quasi-static gas pressures. Specific conclusions drawn from this study and recommendations for future studies will follow.

6.2 Conclusions

Data Review

Experiments conducted from the late 1970's to present indicated that reinforced concrete slabs with stirrups can sustain large support rotations. The data also indicated that design criteria should place significant emphasis on parameters other than simply the scaled range and the type of shear reinforcement. The slab's span-to-thickness ratio, principal reinforcement quantity and spacing, and support conditions are parameters that can, in various combinations of values, be more significant than shear reinforcement in affecting the failure mode and total response of the slab. Specifically, conclusions drawn from the data review include the determination of appropriate shear reinforcement details and associated response limits for military structures subjected to conventional weapons effects. The U.S. Army Corps of Engineers adopted these criteria as they were

derived from this study and are presented in ETL 1110-9-7:

a. Slabs with single-leg stirrups having a 135-degree bend at one end and at least a 90-degree bend at the other end can be designed to sustain support rotations of 12 and 20 degrees for anticipated damage levels categorized as "moderate" and "heavy," respectively. The moderate damage level is that recommended for the protection of personnel and sensitive equipment. Heavy damage means that the slab is near incipient collapse.

b. Limitations for applying the response limits, consistent with the data, include:

(1) The scaled ranged at which the slab is subjected to blast must exceed 0.5 ft/lb$^{1/3}$.

(2) The span-to-effective depth (L/d) ratio must exceed 5.

(3) Principal reinforcement spacing shall never exceed the effective depth (d) of the slab.

(4) Stirrups are required along each principal bar at a maximum spacing of one-half the effective depth (d/2) when the scaled range is less that 2.0 ft/lb$^{1/3}$ and at a maximum spacing equal to the effective depth at larger scaled ranges.

Experimental Investigation

Ultimate Capacity. In the experimental investigation of this study, compressive membrane forces acted to increase the ultimate capacities of the sixteen one-way slabs from approximately 1.2 to 4.0 times the computed Johansen yield-line resistance. It appeared that lacing was slightly more effective than stirrups in

enhancing the ultimate capacities of the slabs. Only for the case of the slabs with a medium ρ value (0.0056) did the slab with stirrups attain a greater ultimate capacity than that with lacing.

The average Δ_A/t ratio (the ratio of midspan deflection occurring at ultimate capacity to the slab thickness) for the slabs was approximately 0.29. There was no consistent pattern to indicate that the Δ_A/t ratio was affected by the construction parameters studied. Consistent with previous work by others, the enhancement in ultimate capacity by compressive membrane forces was greatest for slabs with the smallest ρ, and it decreased as ρ increased. The generally-known compressive membrane theory closely predicted the ultimate capacities of the slabs having the ρ values of 0.0025 and 0.0056 when the experimentally obtained values of Δ_A/t were used; but, a low Δ_A/t value of approximately 0.1 was required for the theory to predict the ultimate capacities of the slabs having a ρ value of 0.0097.

Tensile-Membrane Behavior. Significant spreading of cracking along the length of the slabs did not occur; therefore, significant tensile-membrane behavior did not develop. For the slabs having a ρ value of 0.0025, the tensile-membrane response (and thus the peak reserve capacity) appeared to be best enhanced by lacing. However, for the slabs having a ρ value of 0.0097, the tensile-membrane behavior appeared to be best enhanced by stirrups. The two types of shear reinforcement appeared to be equally effective in the slabs with the medium ρ value of 0.0056. Of the parameters that were varied, the principal reinforcement ratio was the most significant parameter affecting the reserve capacity. The tensile-membrane theory closely predicted the peak

reserve capacities of the slabs with the large ρ value when one-half of the principal steel was considered to be effective. It closely predicted the peak reserve capacities of the slabs with the small ρ value when all of the principal steel was considered to be effective. The peak reserve capacities of the slabs with the medium ρ value were bracketed by the theory when both cases were considered.

As a result of the slabs responding as three-hinge mechanisms, crack width was highly dependent on deflection. Some smoothing (spreading of cracking and formation of a catenary, particularly on the top face) occurred in the slabs with the large ρ value. This smoothing appeared to be greatest for slab no. 5; however, slab no. 5 exhibited the least tendency for tensile membrane behavior. Slab no. 5 did exhibit a significantly more gradual drop in resistance following the ultimate capacity. In general, crack widths were slightly less in the laced slabs than in the slabs with stirrups. Strain gage data indicated that lacing bars yielded at lower pressure levels and smaller slab deflections than did the vertical stirrups, indicating that the lacing was mobilized earlier in making a contribution to a slab's response. However, the overall responses of the laced and stirrup slabs were very similar, differing little in resistance values. Other than for slabs no. 5 and 15, the companion pairs of laced and stirrup slabs exhibited load-deflection curves with very similar shapes.

Overall Response. This investigation indicated that one-way slabs typical of protective construction (equal top and bottom steel, restrained at ends) are susceptible to shear failure when

reinforced with approximately 0.5 percent or more principal reinforcement, but no shear reinforcement. Shear reinforcement may not be needed to insure a flexural failure mode in slabs with approximately 0.25 percent principal reinforcement. Support rotations from approximately 20 to 30 degrees were achieved by the fourteen slabs that did not incur shear failure.

No significant differences were observed in the behavior of the slabs with lacing bars and the slabs with stirrups that were experimentally evaluated in this study. The slight increase in ultimate capacity for laced slabs cannot justify the complications and expense associated with the construction of laced slabs. Single-leg stirrups with a 90-degree bend on one end and a 135-degree bend on the other are sufficient for preventing shear failure and for enhancing the reserve capacity to the same level (or, as in some cases of this study, to a higher level) as lacing bars. The experiments showed that, for slabs with principal steel spaced at approximately one-half to two-thirds of d and shear reinforcement spaced less than d, variations in the principal reinforcement ratio has a significantly greater effect on slab response than does the type and ratio of the shear reinforcement.

Application to Design Criteria. The more ductile response and improved large-deflection behavior that one would expect, based on TM 5-1300, from a laced slab over a slab with stirrups did not occur in this study. The damage levels experienced by the slabs in this study fall into the heavy damage category of ETL 1110-9-7. The data from these experiments support the response limits given in the ETL as being aggressive, yet adequate, design values for slabs of military protective structures that can allow

the occurrence of heavy damage, but not collapse. Additionally, this study indicated that design criteria concerning shear reinforcement and slab response limits in TM 5-1300 are overly restrictive. Although the experiments conducted in this study do not necessarily demonstrate the response of the slabs to any possible blast environment that may occur in an explosives manufacturing/storage facility, they are at least representative of slabs loaded by the slower rising quasi-static pressure that accompanies an internal detonation. Recognition of this conclusion alone will result in a significant increase in the allowable response limits (on the order of those given in ETL 1110-9-7) of some wall slabs within such facilities. In addition, by combining the findings of the experiments conducted during this investigation with the parameter study (data review), one may be reasonably confident that the failure modes and response limits exhibited by the slabs will be duplicated in a direct blast pressure loading that results from a detonation at a scaled range greater than 2.0 ft/lb$^{1/3}$ and possibly as low as 1.0 ft/lb$^{1/3}$.

6.2 Recommendations

This investigation merged together an understanding of the history of the development of current design criteria with new data that showed the similar effects of lacing bars and stirrups. Modifications to the shear reinforcement criteria and allowable response limits of TM 5-1300 should be initiated. As a minimal revision, the allowable response limit for slabs that contain stirrups and are subjected to blast at scaled ranges greater than 1.0 ft/lb$^{1/3}$ should be increased to that of laced slabs (12 degrees

of support rotation). Experiments using dynamic loading conditions should be conducted to validate the findings of this study and to further study the effects of lacing and stirrups in close-in blast environments. Additionally, this study should be extended to slabs with other L/d ratios, particularly "deep" ($L/d < 5$) slabs.

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APPENDIX
DATA

A.1 Instrumentation Data

The electronically recorded data for all sixteen experiments are presented in this appendix. All of the strain gage and the deflection gage readings were plotted against the readings of each pressure transducer (P-1 and P-2). For the plots presented herein, the strain and deflection measurements versus the readings of only one of the pressure gages are shown.

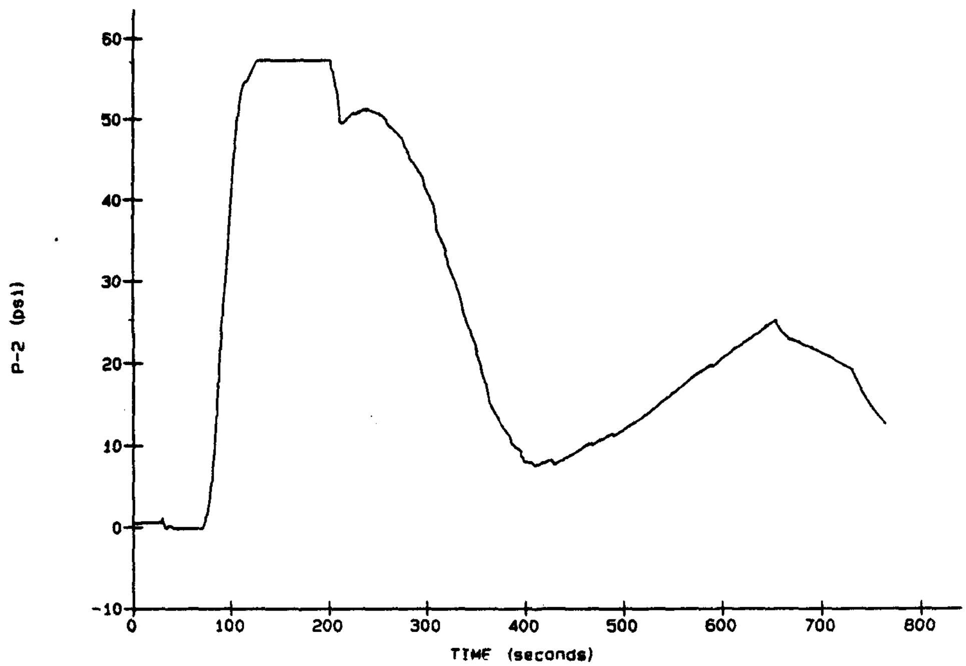
In general, the quality of the recovered data was good. As often occurs when many strain gages are embedded in concrete, several strain gages did not function properly. These included: SL-4 in slab no. 8; SL-1 and SL-6 in slab no. 9; SS-1 in slab no. 10; ST-1 in slab no. 11; and SS-3 in slab no. 14. All but one of these malfunctioning gages were located on shear reinforcement. One was located on a top principal reinforcement bar.

The data were considerably clean, but there were some instances of noisy records such as the unloading phase recorded by gage D-2 of slab no. 3. The noise in that particular record is likely due to the effects of water entering the deflection gage housing following rupture of the membrane that covered the slab. Noisy records primarily included that of strain gage readings when measured values were considerably lower than the calibration values.

As a result a of misunderstanding of the over-range capabilities of a new digital data acquisition system, the data from slab no. 1 (the first of the series) was clipped at an overpressure level of approximately 57 psi. It appears from the

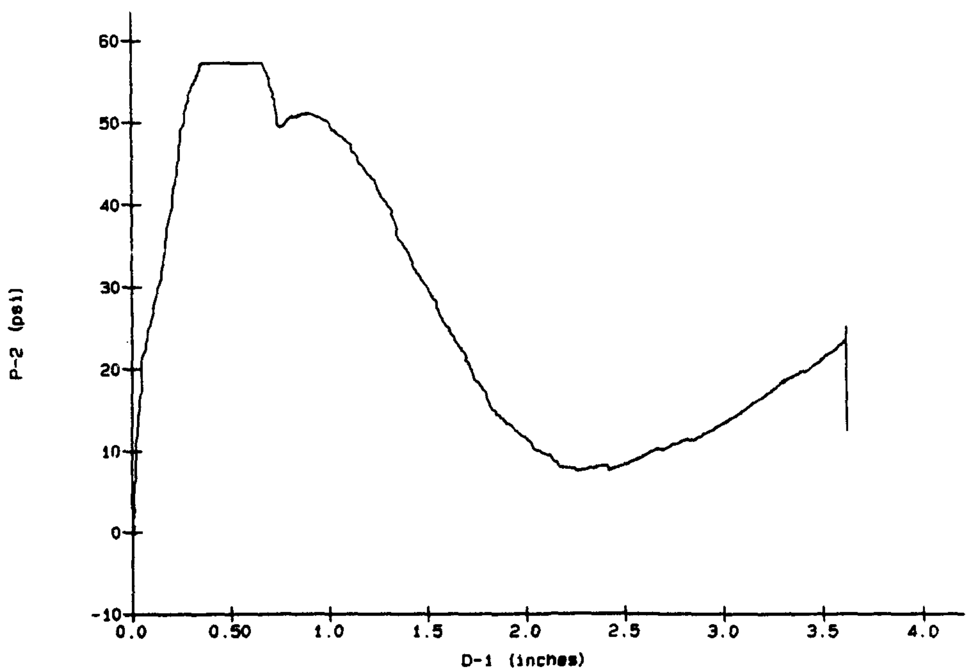
shape of the load-deflection curve that the peak overpressure applied to slab no. 1 was in the range of approximately 60 to 65 psi. The plots for midspan deflection measurements (D-1 gage readings) of slab nos. 12, 14, and 16 indicate that the cable from the deflection transducer became unattached to the slab surface during the experiments on these three slabs. However, the quarterspan deflections were successfully recorded throughout the experiments.

SLAB 1



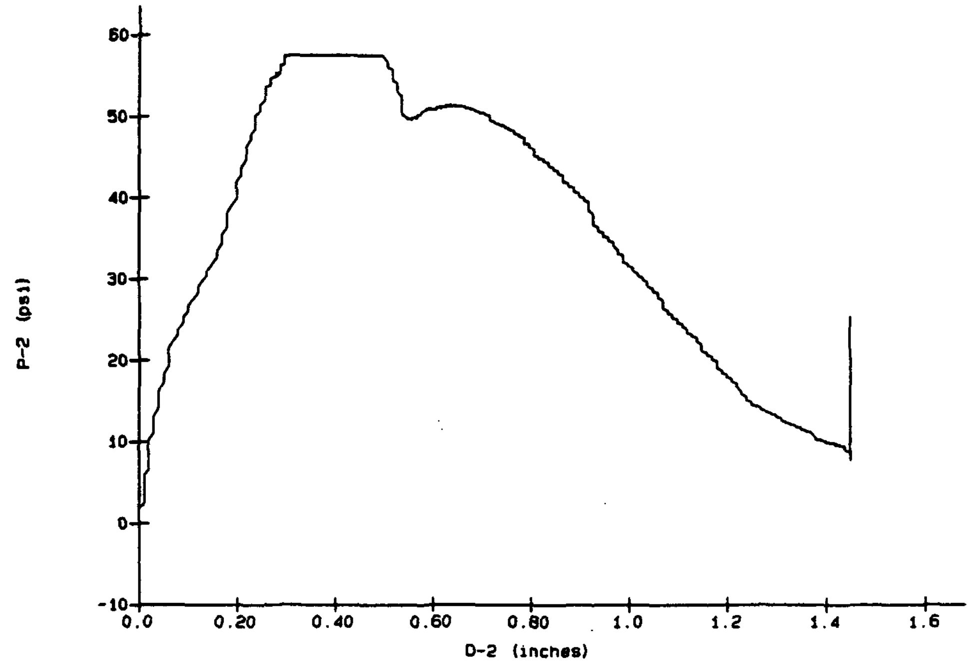
60
50
40
30
20
10
0
-10
P-2 (psi)
0
100
200
300
400
500
600
700
800
TIME (seconds)

SLAB 1



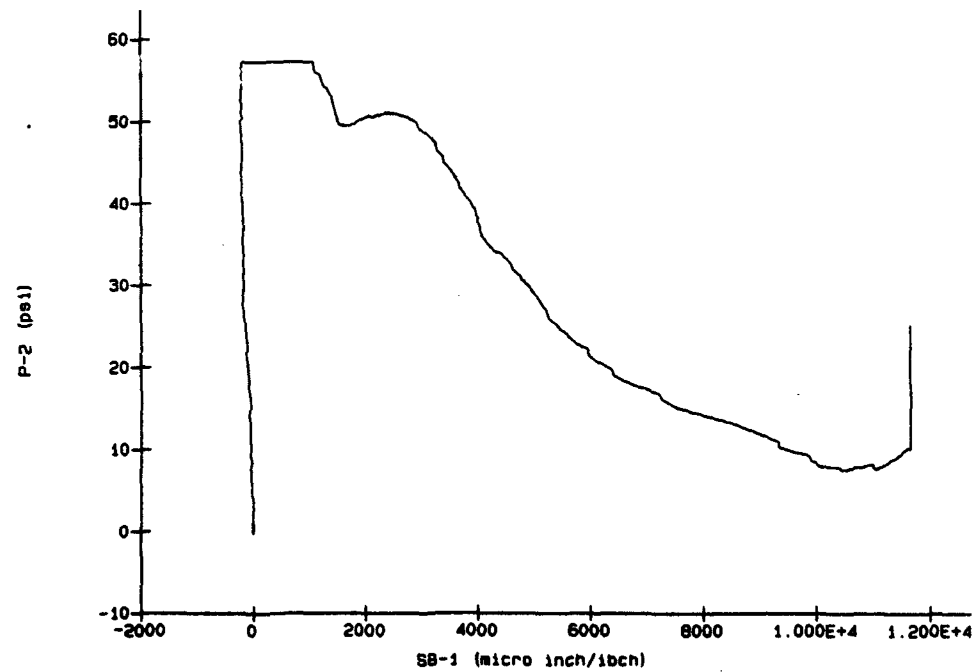
60
50
40
30
20
10
0
-10
P-2 (psi)
0.0
0.50
1.0
1.5
2.0
2.5
3.0
3.5
4.0
D-1 (inches)

SLAB 1

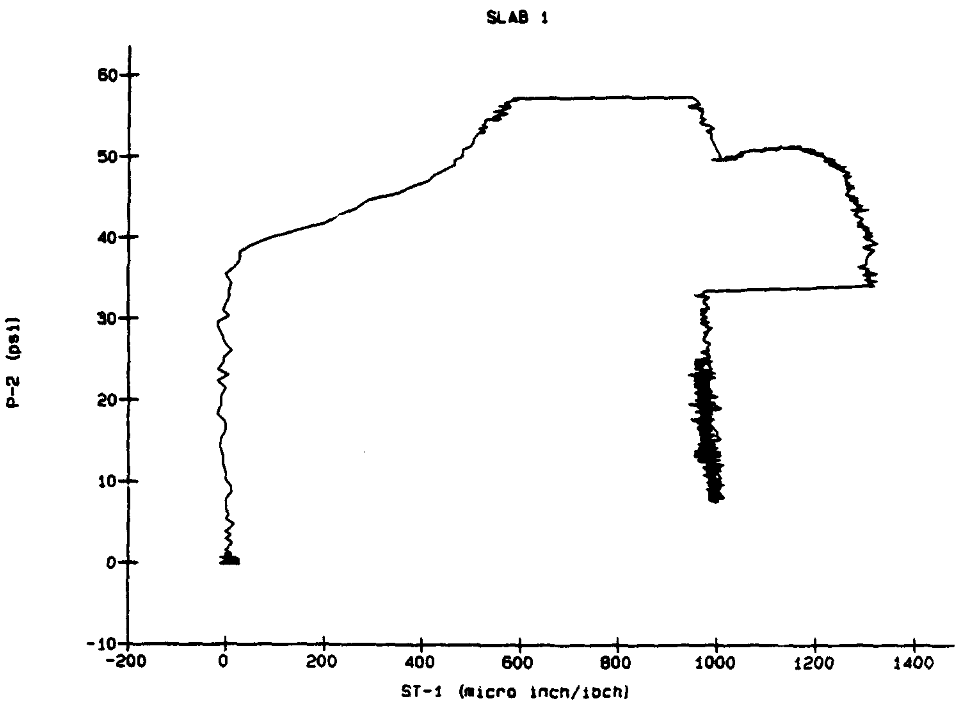


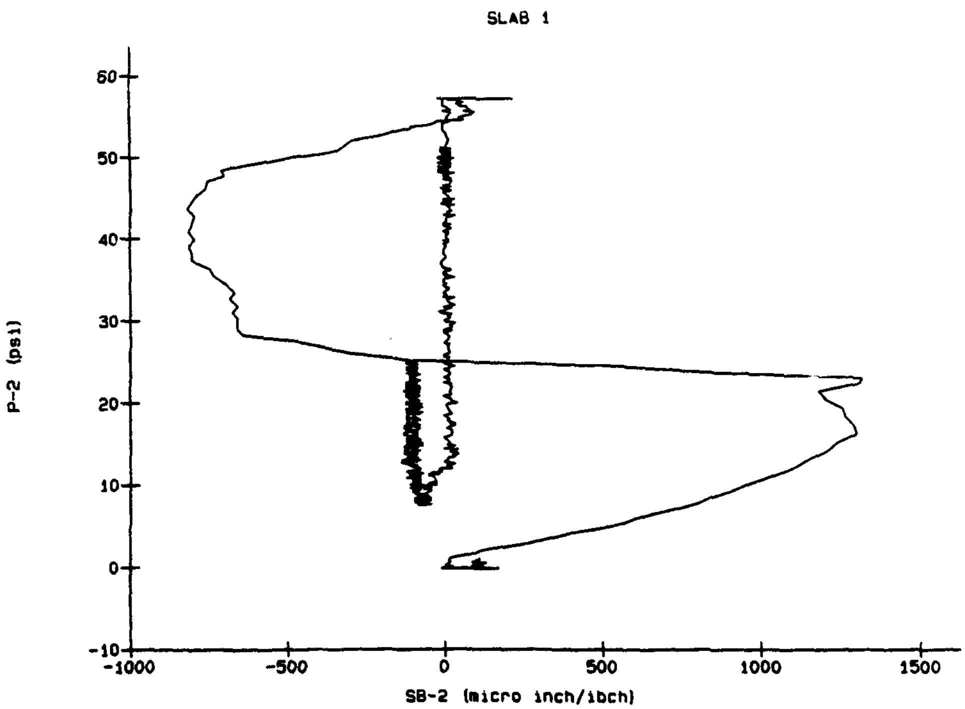
P-2 (psi)
60
50
40
30
20
10
0
-10
0.0
0.20
0.40
0.60
0.80
1.0
1.2
1.4
1.6
D-2 (inches)

SLAB 1

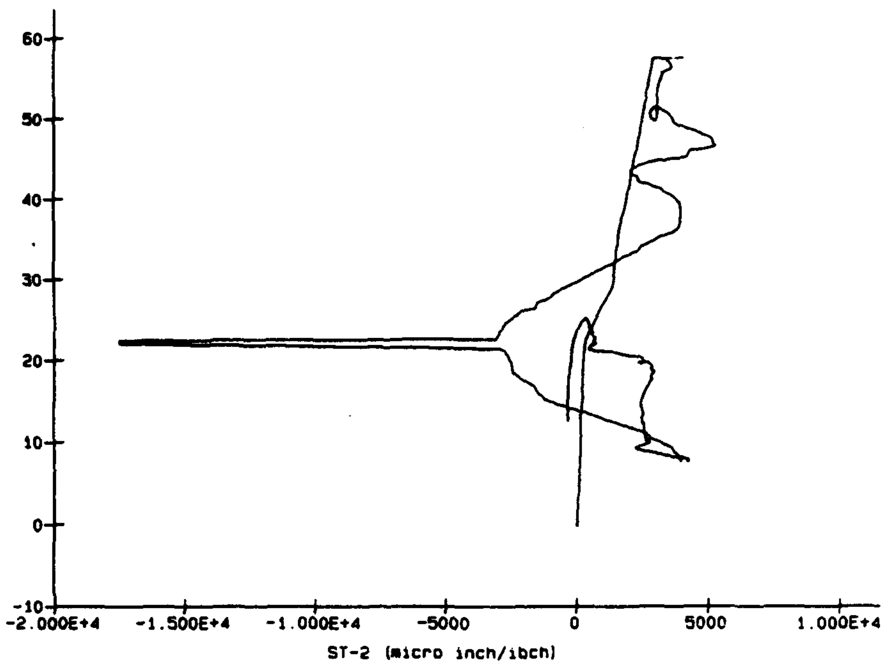


P-2 (psi)
60
50
40
30
20
10
0
-10
-2000
0
2000
4000
6000
8000
1.000E+4
1.200E+4
SB-1 (micro inch/ibch)

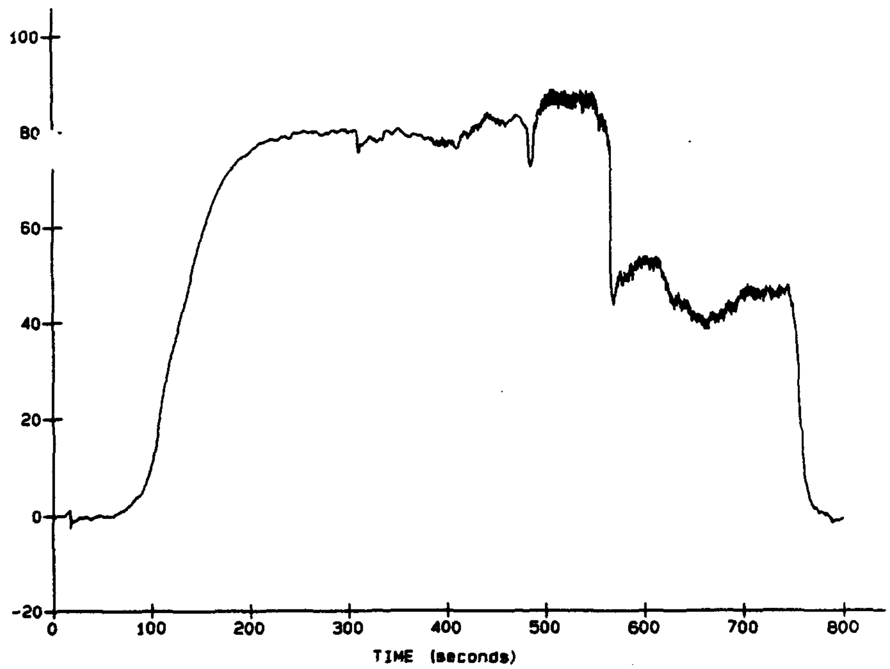

SLAB 1
P-2 (psi)
60
50
40
30
20
10
0
-10
-200
0
200
400
600
800
1000
1200
1400
ST-1 (micro inch/inch)

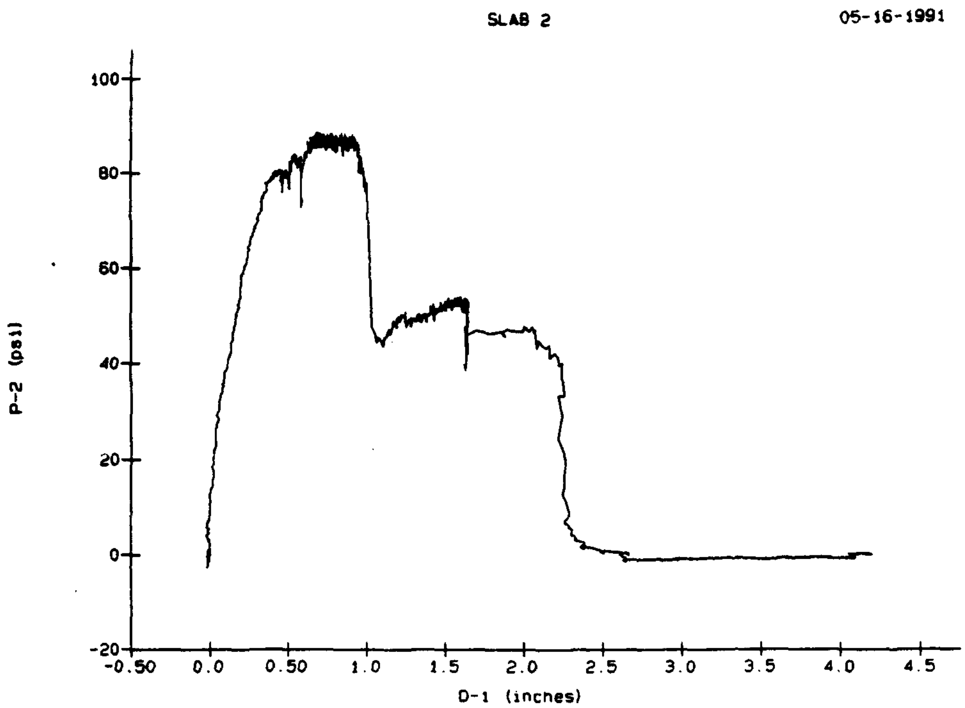

SLAB 1
P-2 (psi)
60
50
40
30
20
10
0
-10
-1000
-500
0
500
1000
1500
SB-2 (micro inch/inch)

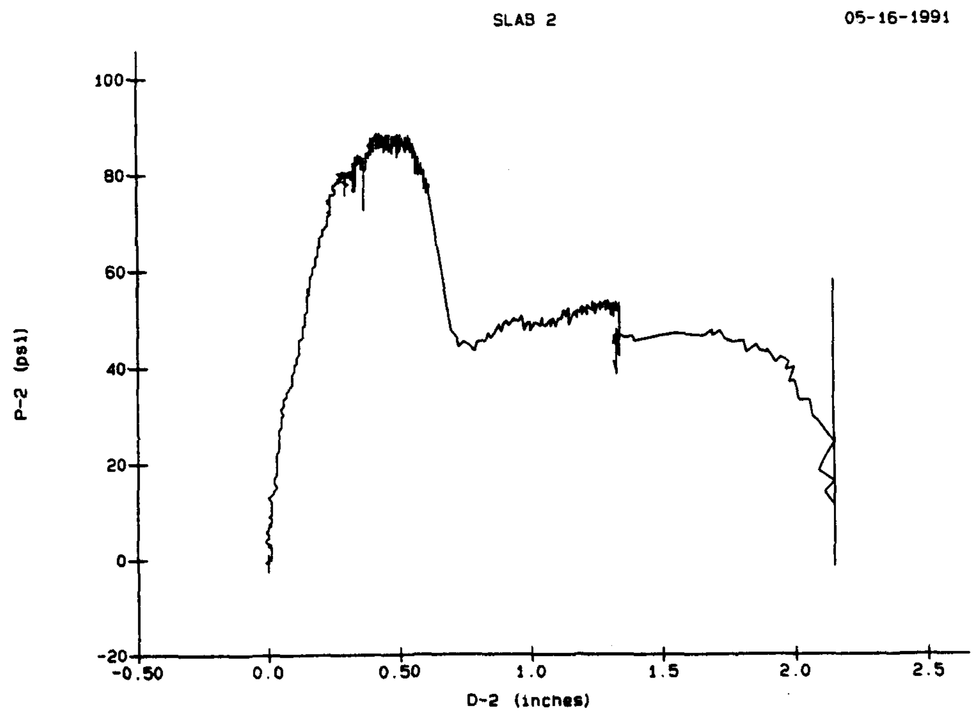
SLAB 1

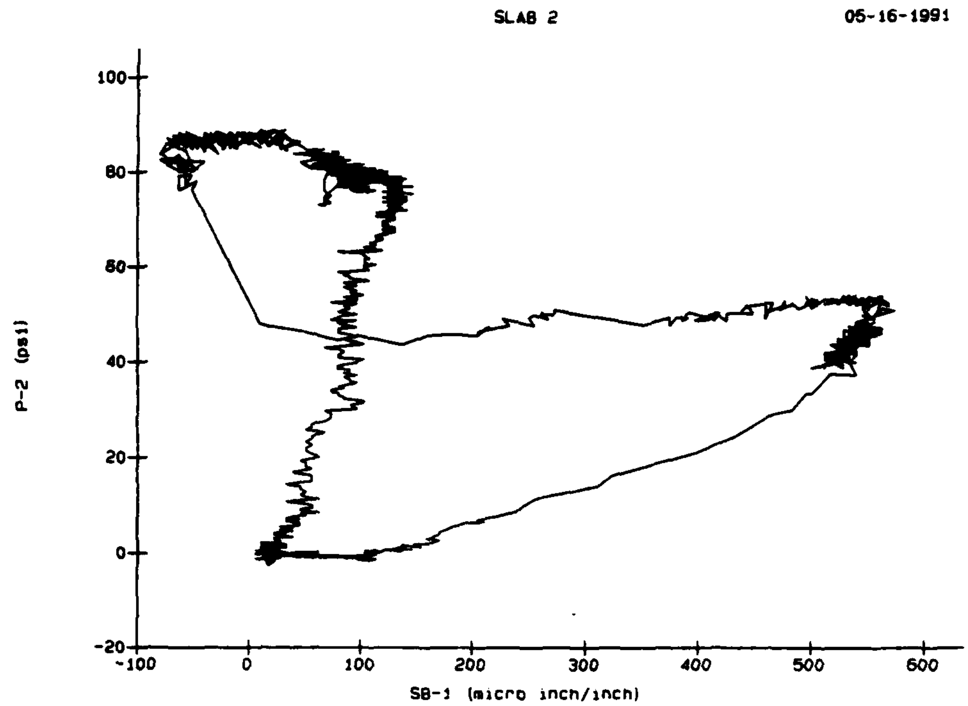

60
50
40
30
20
10
0
-10
P-2 (psi)
-2.000E+4
-1.500E+4
-1.000E+4
-5000
0
5000
1.000E+4
ST-2 (micro inch/ibch)

SLAB 2

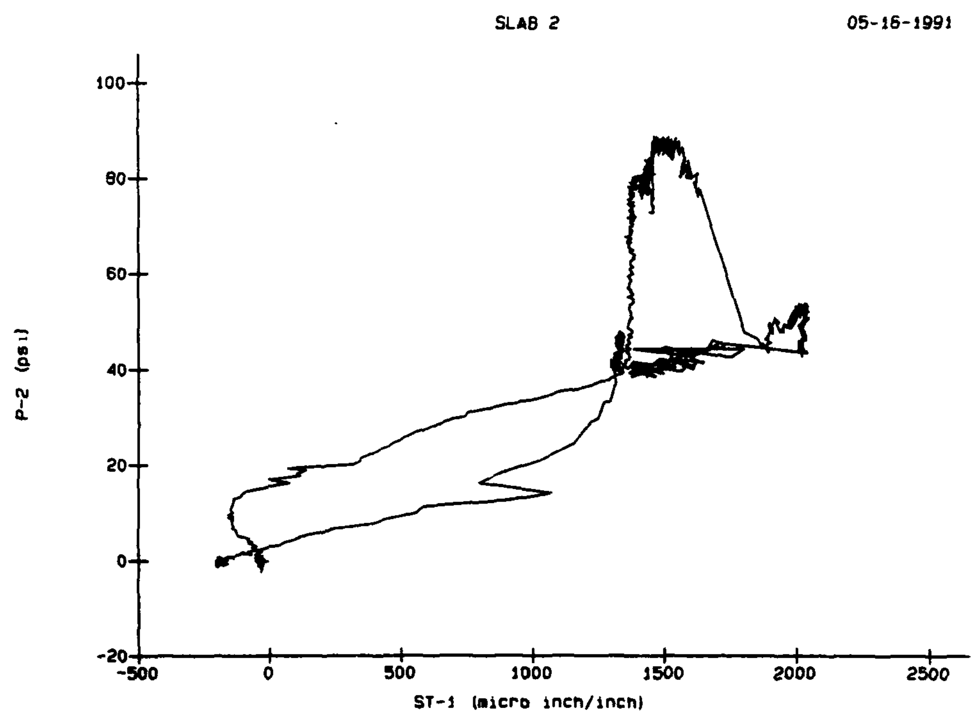

100
80
60
40
20
0
-20
P-2 (psi)
0
100
200
300
400
500
600
700
800
TIME (seconds)


SLAB 2
05-16-1991
P-2 (psi)
100
80
60
40
20
0
-20
-0.50
0.0
0.50
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
D-1 (inches)

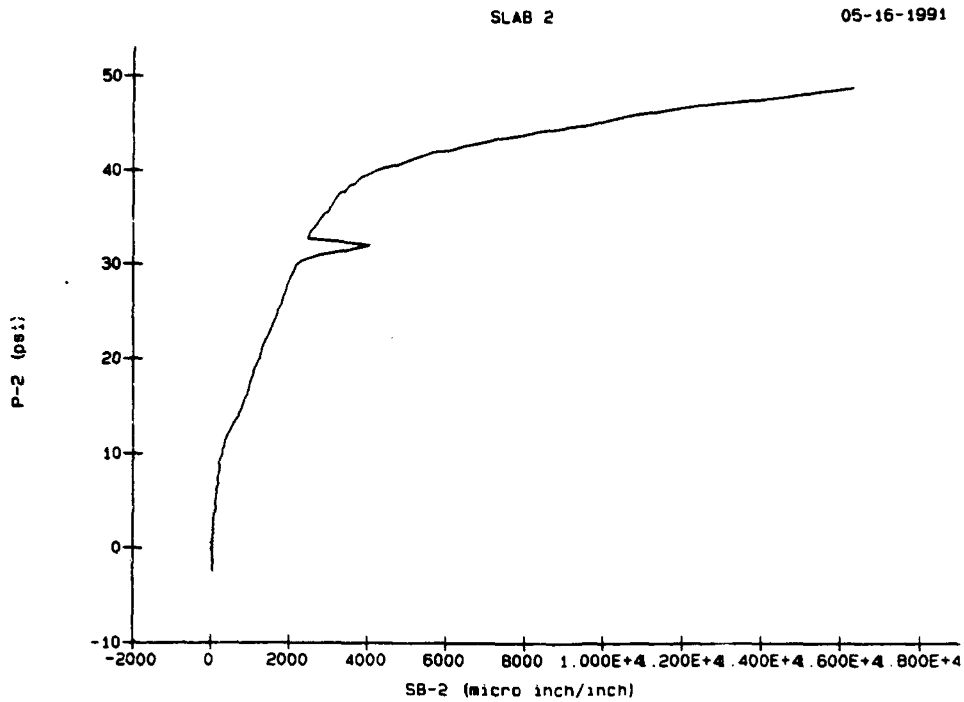

SLAB 2
05-16-1991
P-2 (psi)
100
80
60
40
20
0
-20
-0.50
0.0
0.50
1.0
1.5
2.0
2.5
D-2 (inches)

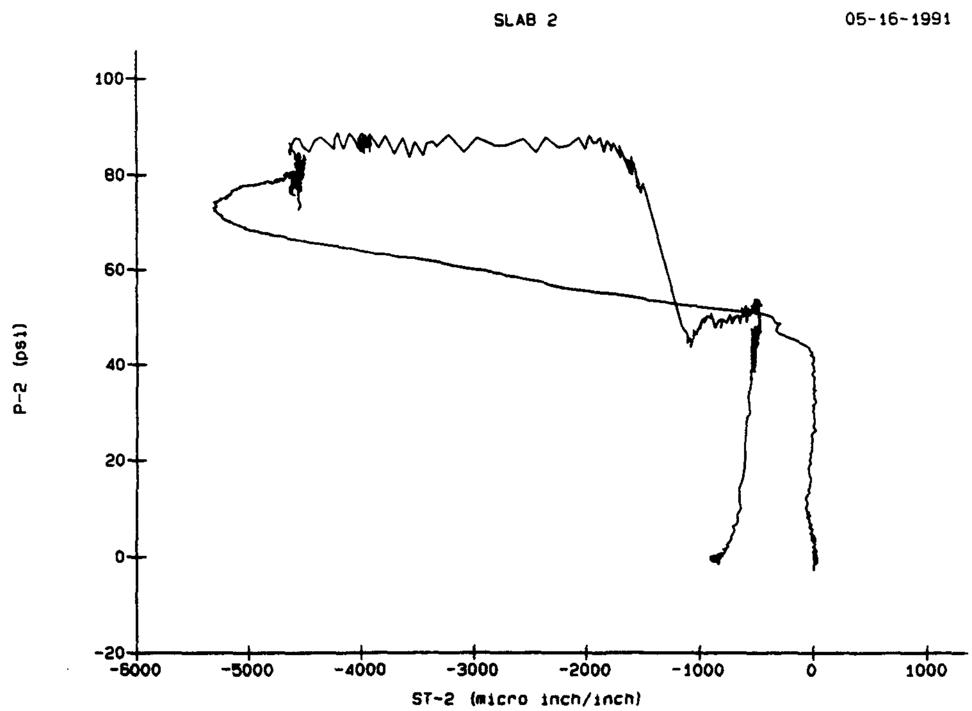


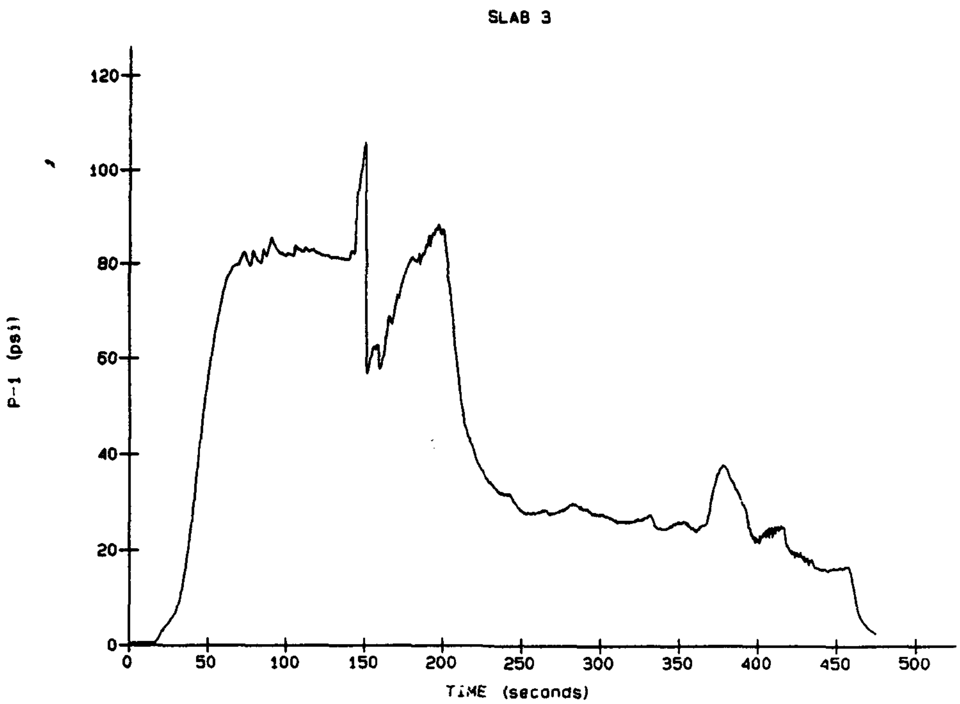
SLAB 2
05-16-1991
P-2 (psi)
100
80
60
40
20
0
-20
-100
0
100
200
300
400
500
600
SB-1 (micro inch/inch)



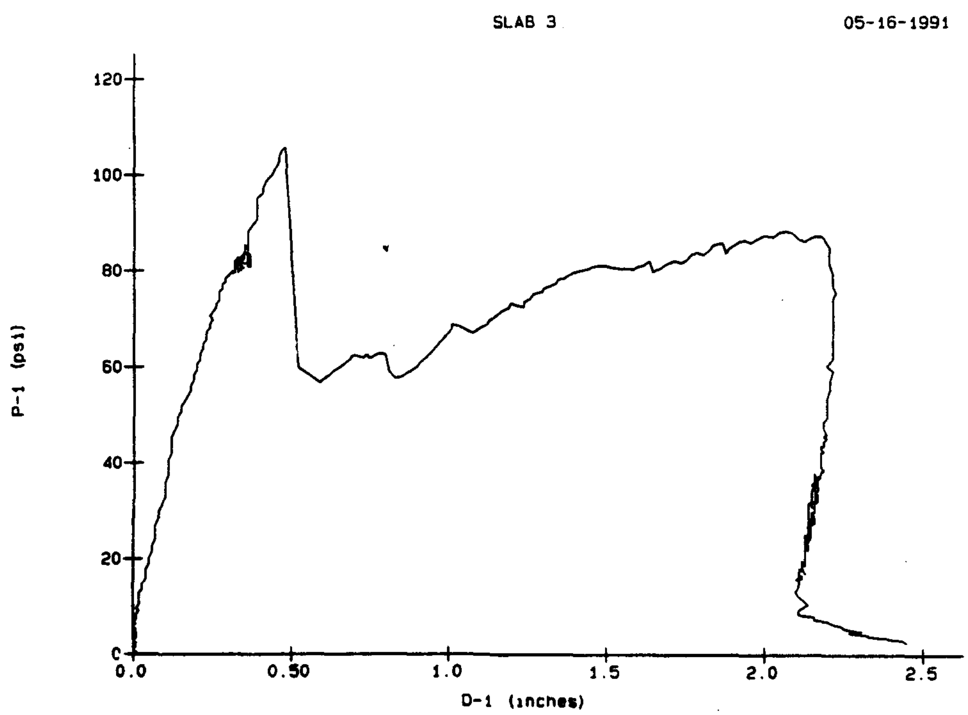
SLAB 2
05-16-1991
P-2 (psi)
100
80
60
40
20
0
-20
-500
0
500
1000
1500
2000
2500
ST-1 (micro inch/inch)


SLAB 2
05-16-1991
50
40
30
20
10
0
-10
-2000
0
2000
4000
6000
8000
1.000E+4
.200E+4
.400E+4
.600E+4
.800E+4
SB-2 (micro inch/inch)
P-2 (psi)

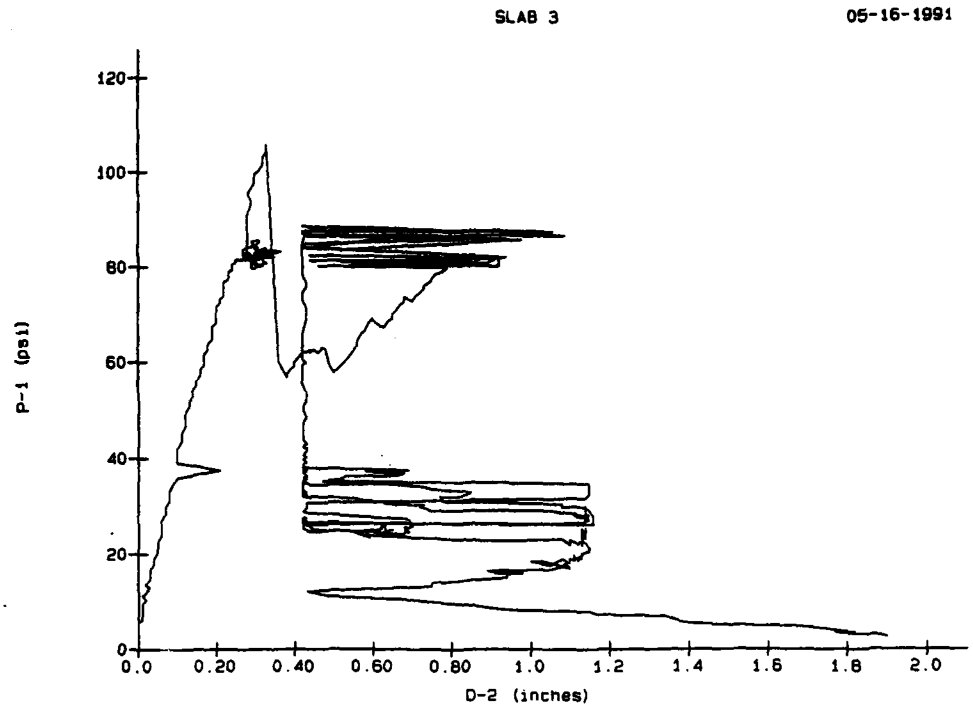

SLAB 2
05-16-1991
100
80
60
40
20
0
-20
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
ST-2 (micro inch/inch)
P-2 (psi)

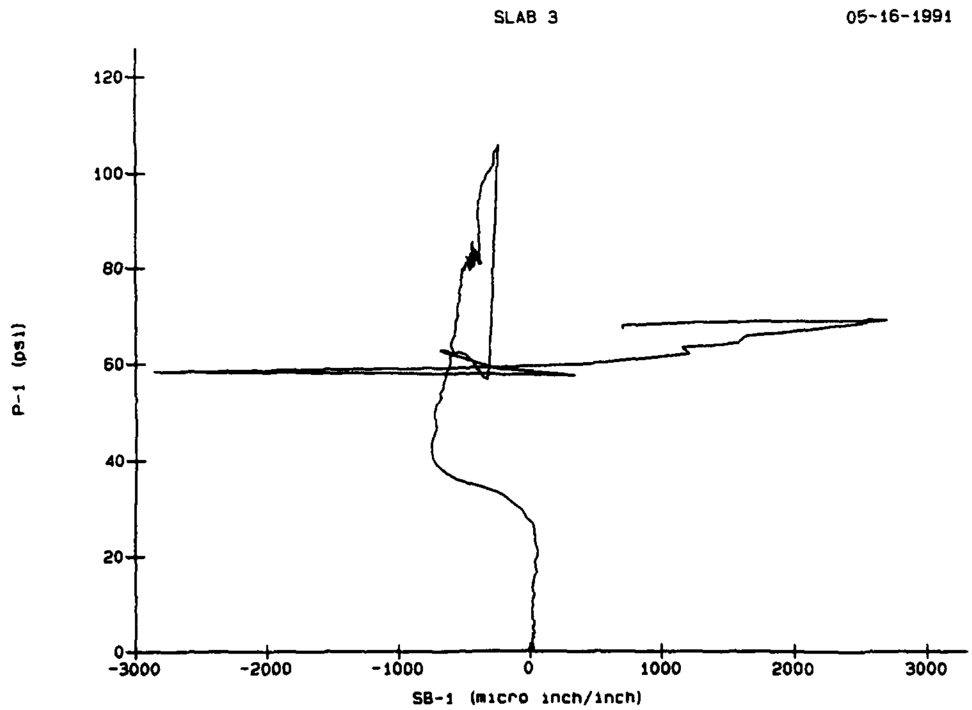


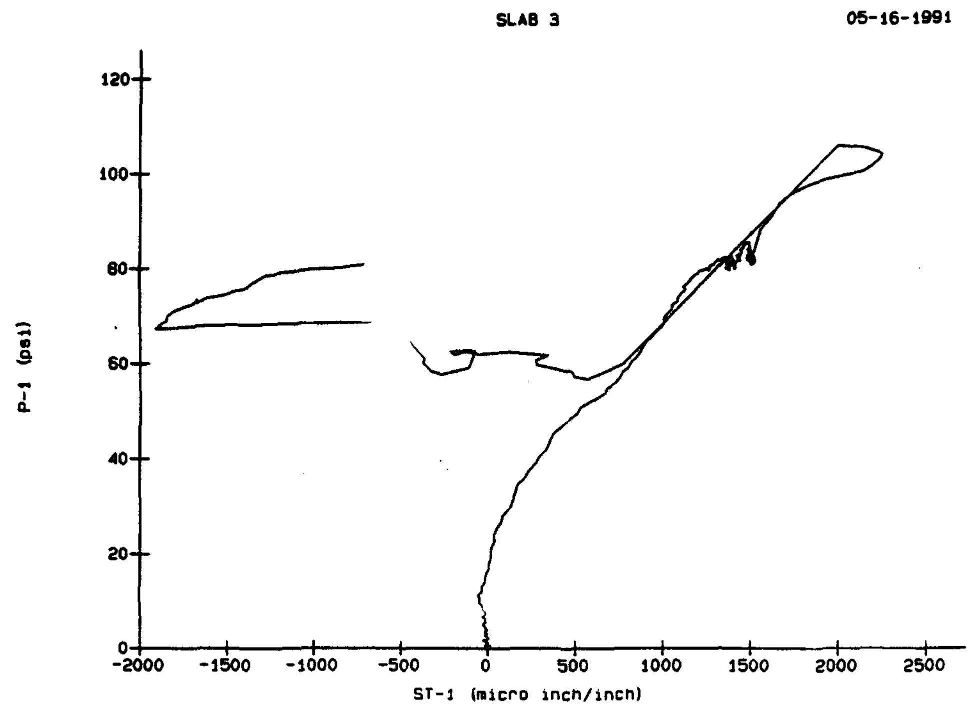
SLAB 3
P-1 (psi)
TIME (seconds)

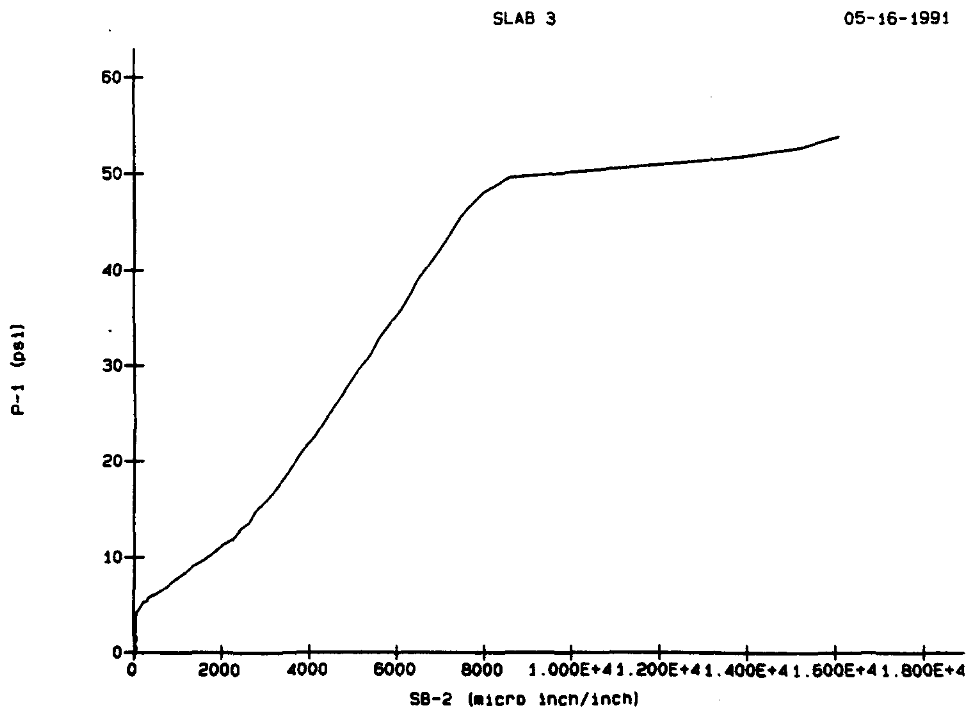


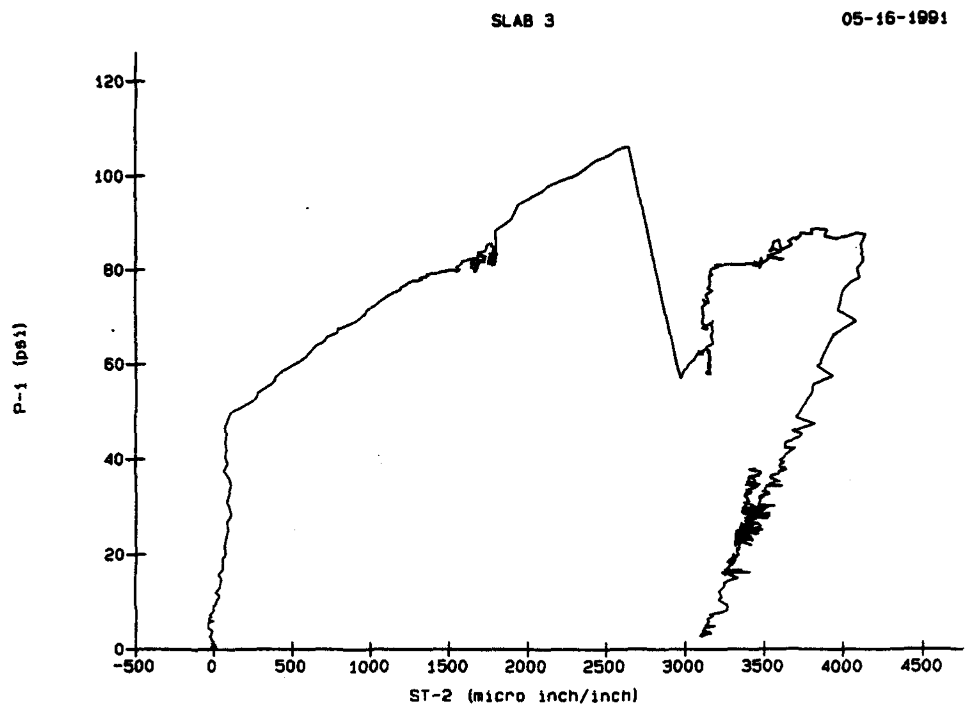
SLAB 3
05-16-1991
P-1 (psi)
D-1 (inches)


SLAB 3
05-16-1991
P-1 (psi)
0
20
40
60
80
100
120
D-2 (inches)
0.0
0.20
0.40
0.60
0.80
1.0
1.2
1.4
1.6
1.8
2.0

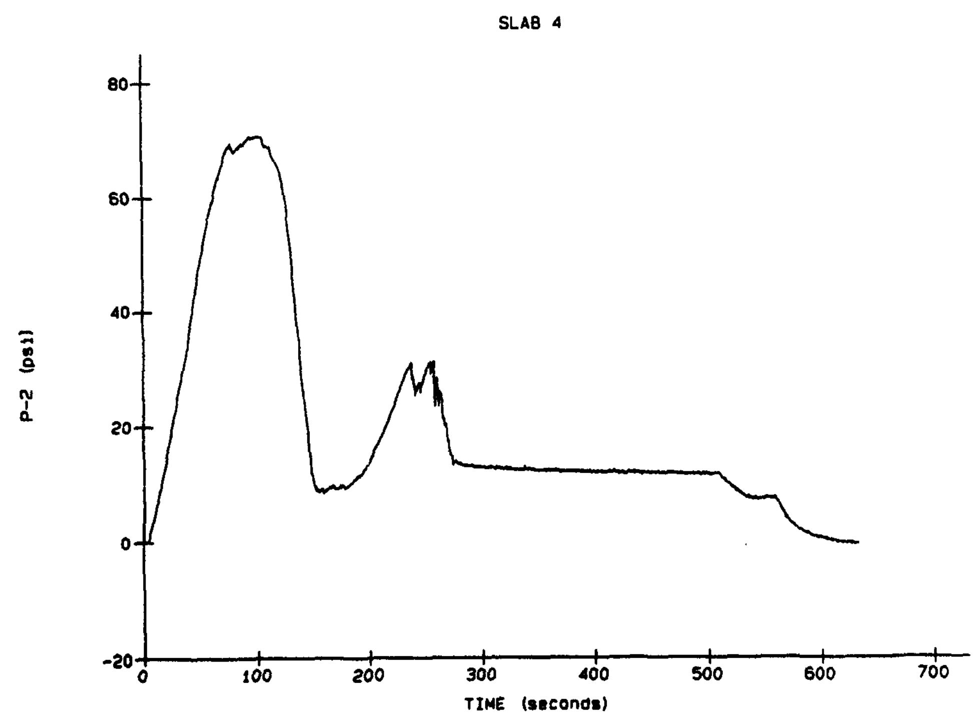

SLAB 3
05-16-1991
P-1 (psi)
0
20
40
60
80
100
120
SB-1 (micro inch/inch)
-3000
-2000
-1000
0
1000
2000
3000


SLAB 3
05-16-1991
P-1 (psi)
ST-1 (micro inch/inch)
120
100
80
60
40
20
0
-2000
-1500
-1000
-500
0
500
1000
1500
2000
2500

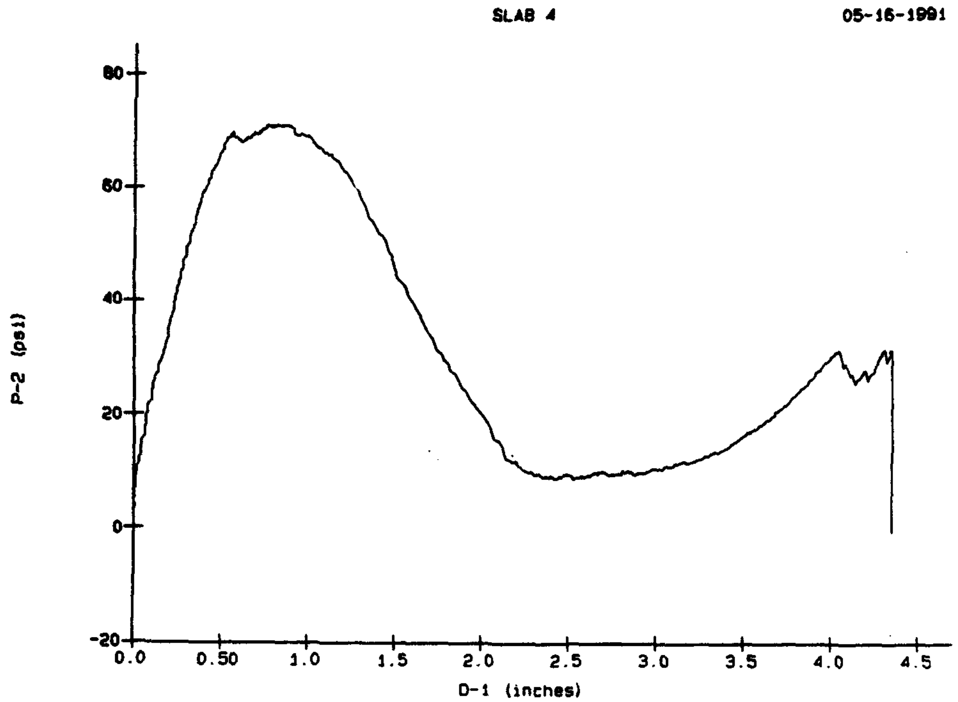

SLAB 3
05-16-1991
P-1 (psi)
SB-2 (micro inch/inch)
60
50
40
30
20
10
0
0
2000
4000
6000
8000
1.000E+4
1.200E+4
1.400E+4
1.600E+4
1.800E+4

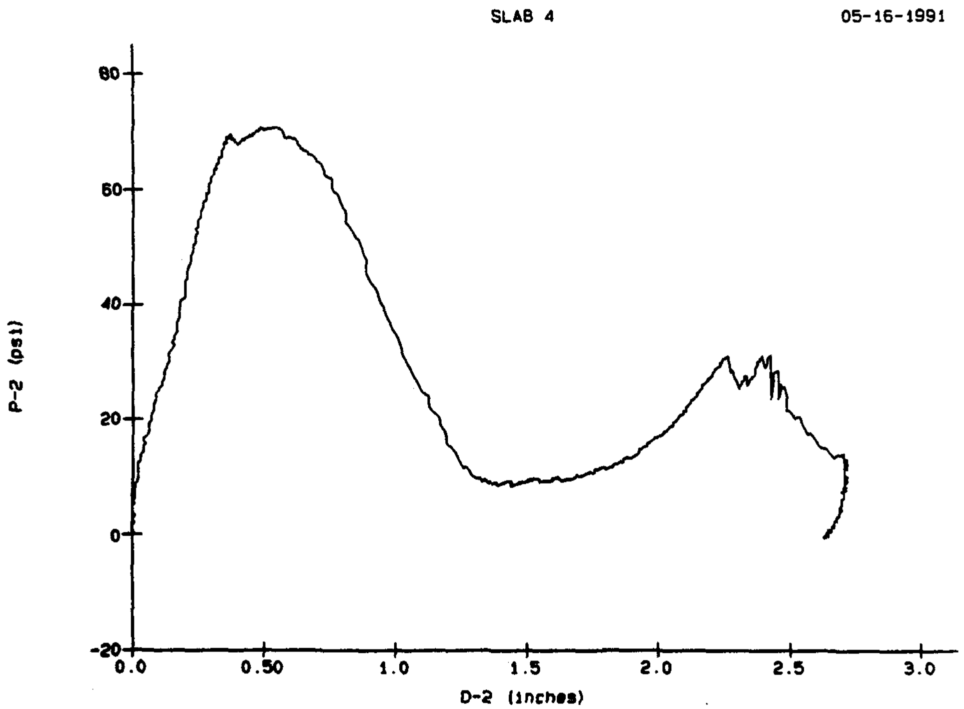


SLAB 3
05-16-1991
P-1 (psi)
ST-2 (micro inch/inch)

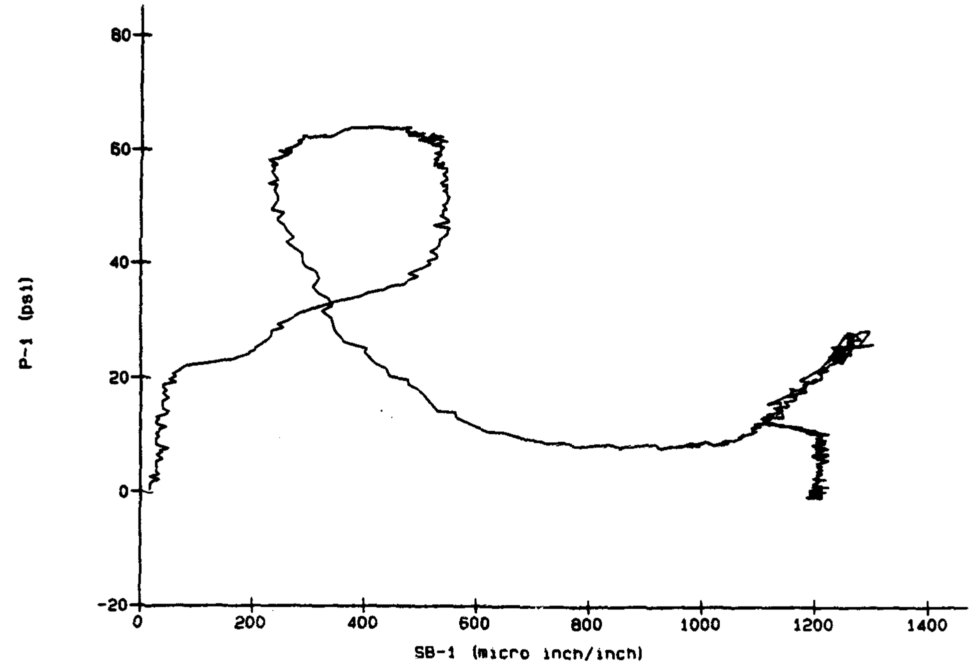


SLAB 4
P-2 (psi)
TIME (seconds)


SLAB 4
05-16-1991
80
60
40
20
0
-20
P-2 (psi)
0.0
0.50
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
D-1 (inches)

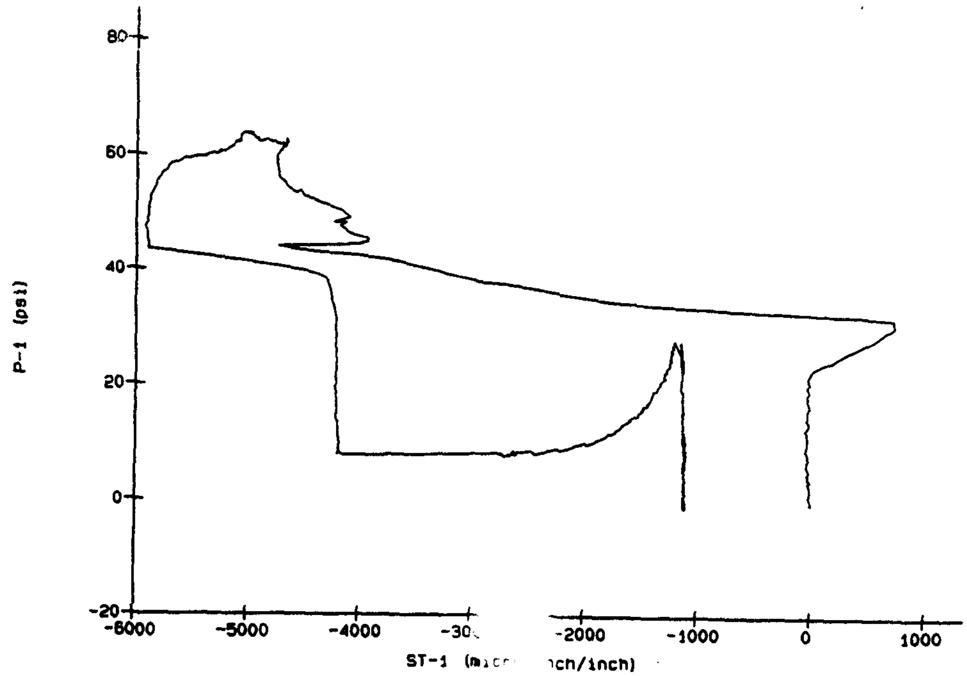

SLAB 4
05-16-1991
80
60
40
20
0
-20
P-2 (psi)
0.0
0.50
1.0
1.5
2.0
2.5
3.0
D-2 (inches)

SLAB 4



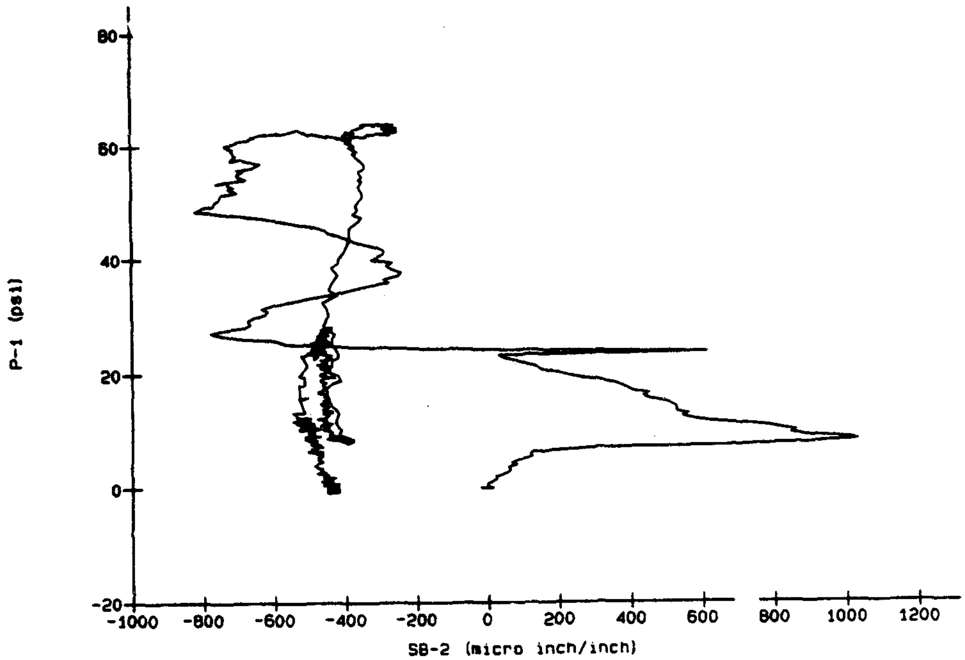
80
60
40
20
0
-20
P-1 (psi)
0
200
400
600
800
1000
1200
1400
SB-1 (micro inch/inch)

SLAB 4



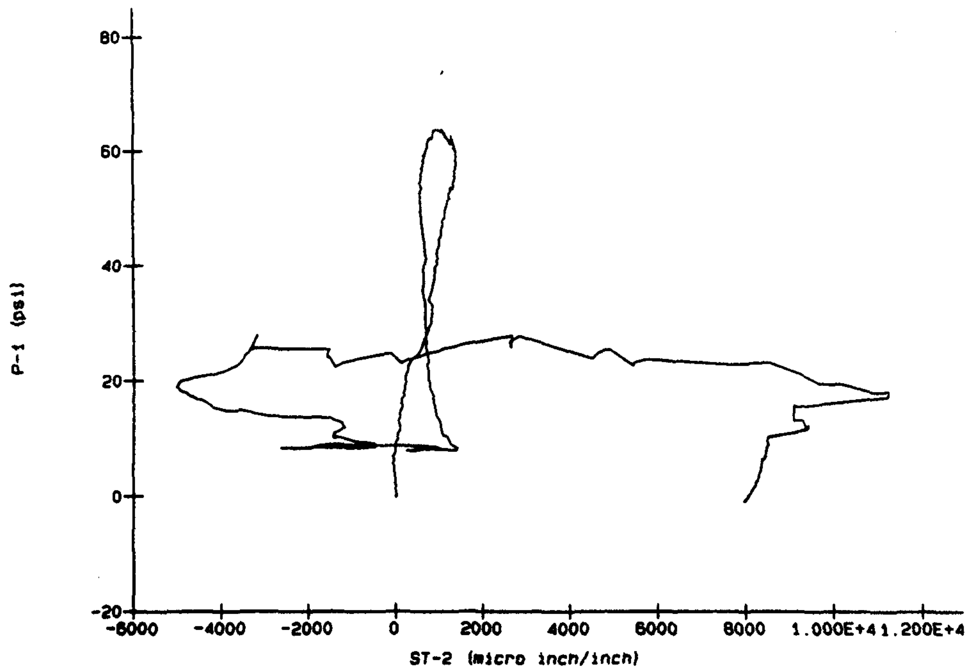
80
60
40
20
0
-20
P-1 (psi)
-6000
-5000
-4000
-2000
-1000
0
1000

SLAB 4



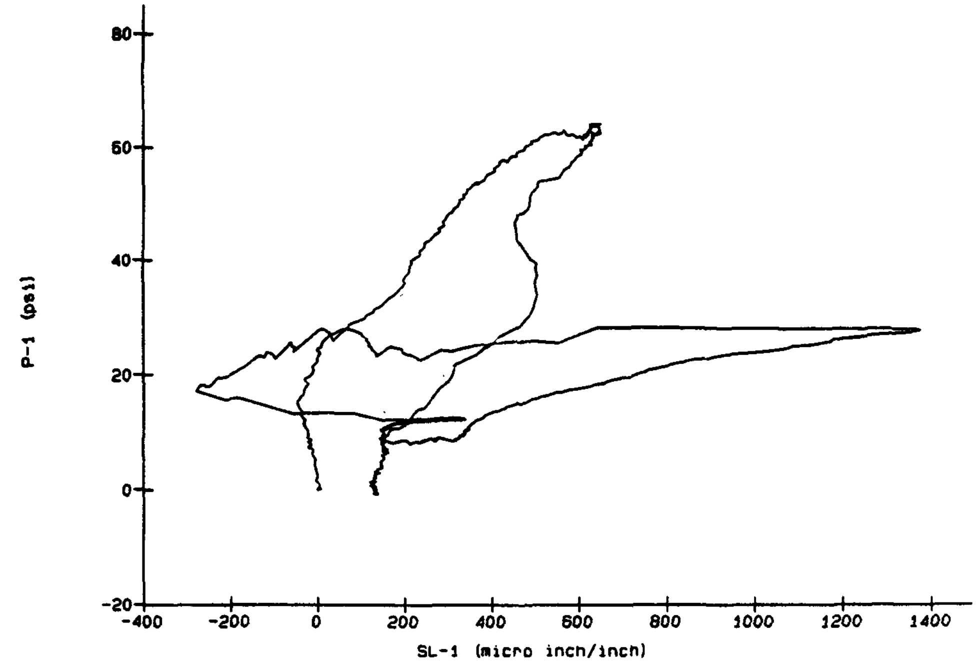
P-1 (psi)
80
60
40
20
0
-20
-1000
-800
-600
-400
-200
0
200
400
600
800
1000
1200
SB-2 (micro inch/inch)

SLAB 4



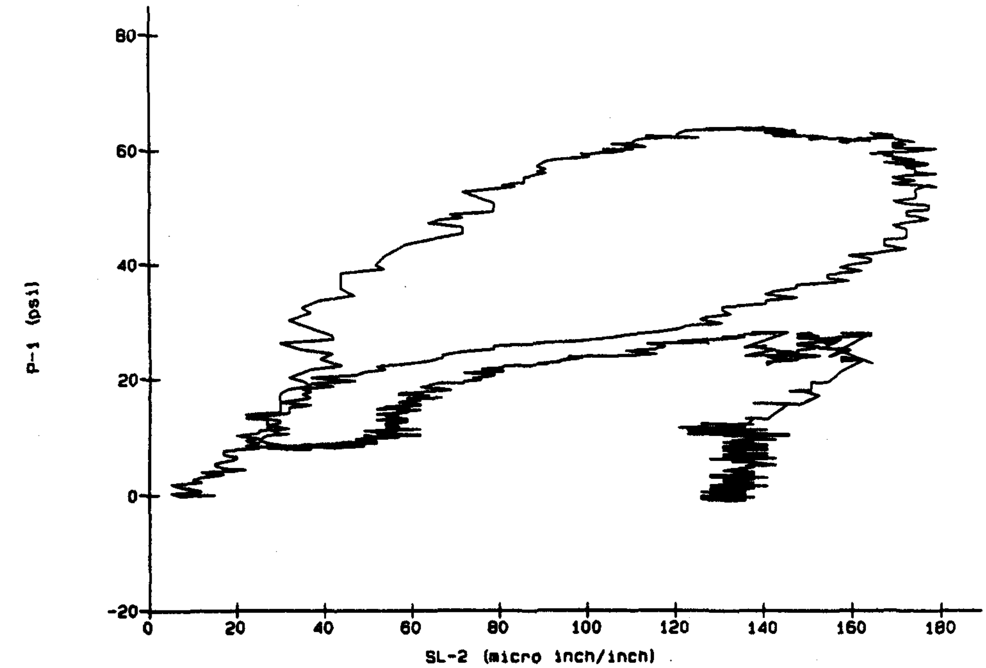
P-1 (psi)
80
60
40
20
0
-20
-6000
-4000
-2000
0
2000
4000
6000
8000
1.000E+4
1.200E+4
ST-2 (micro inch/inch)

SLAB 4

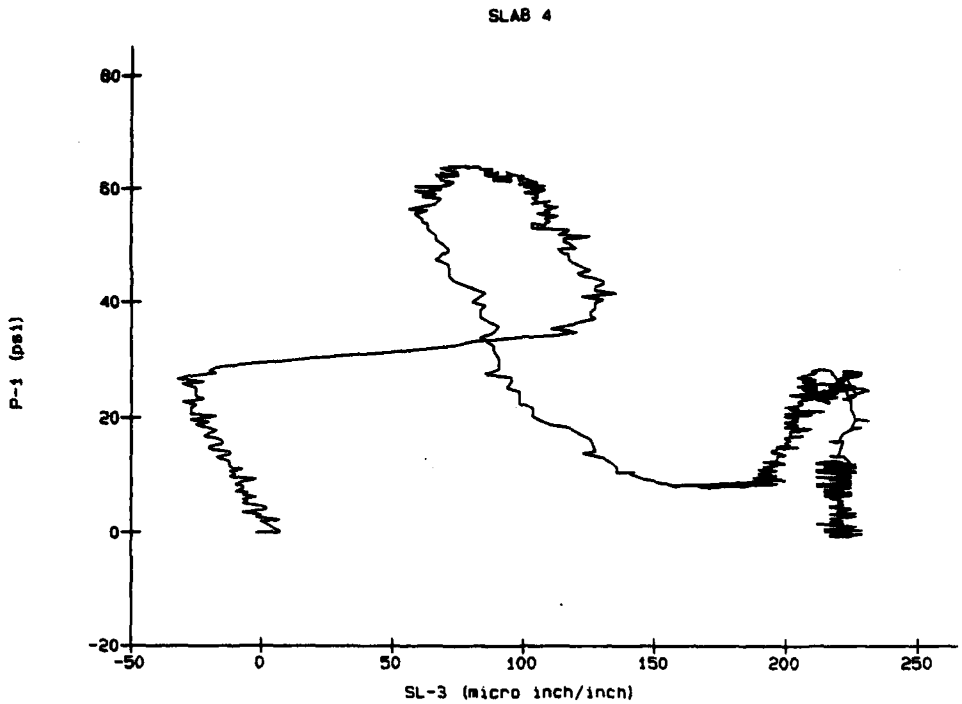


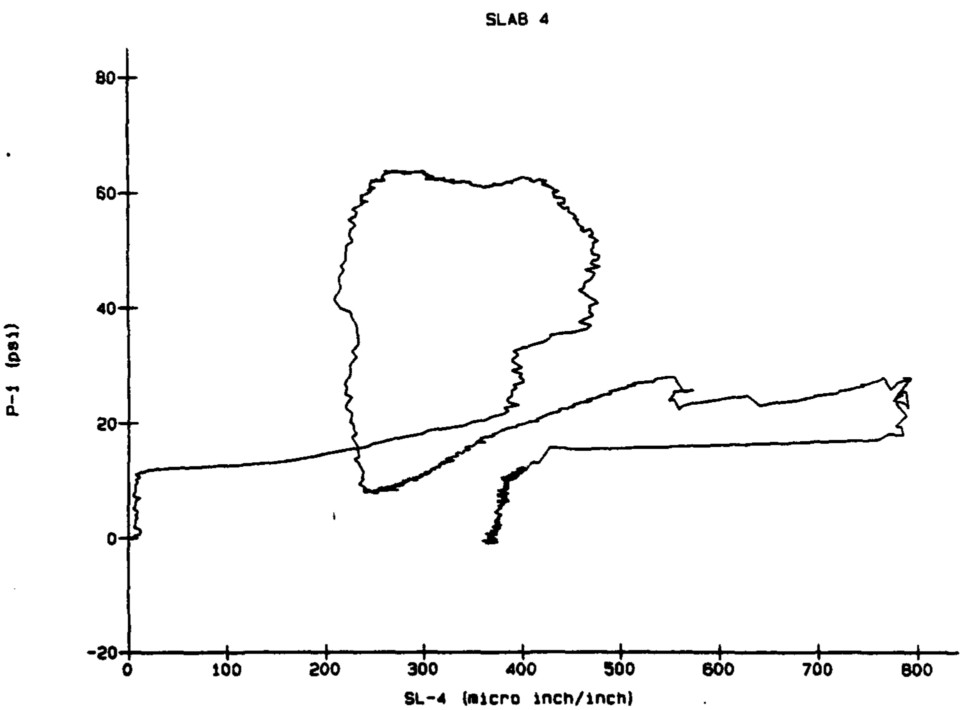
80
60
40
20
0
-20
P-1 (psi)
-400
-200
0
200
400
600
800
1000
1200
1400
SL-1 (micro inch/inch)

SLAB 4

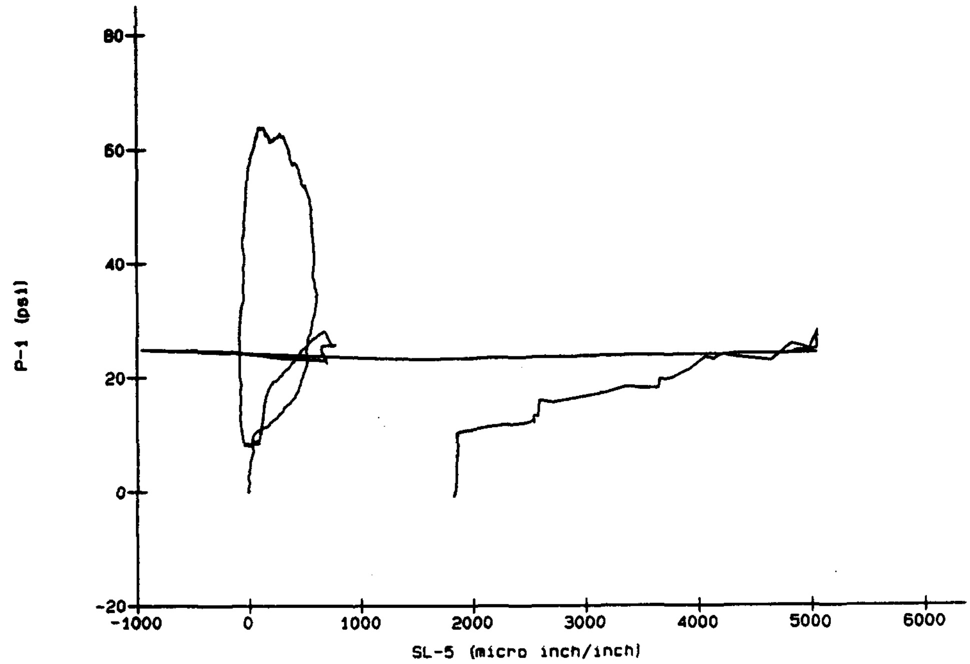


80
60
40
20
0
-20
P-1 (psi)
0
20
40
60
80
100
120
140
160
180
SL-2 (micro inch/inch)


SLAB 4
P-1 (psi)
80
60
40
20
0
-20
-50
0
50
100
150
200
250
SL-3 (micro inch/inch)

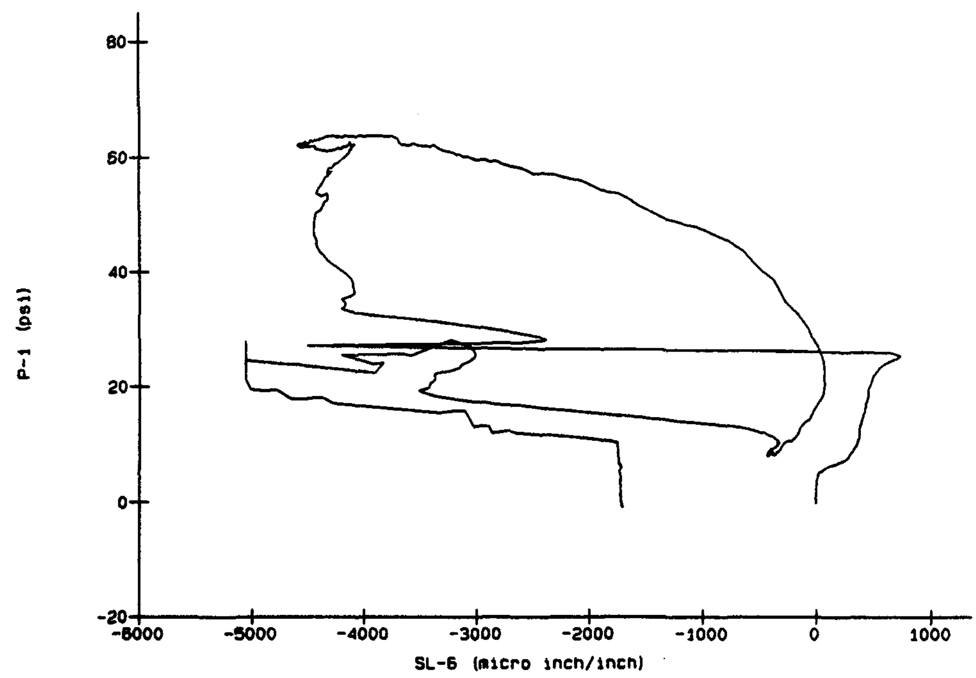

SLAB 4
P-1 (psi)
80
60
40
20
0
-20
0
100
200
300
400
500
600
700
800
SL-4 (micro inch/inch)

SLAB 4

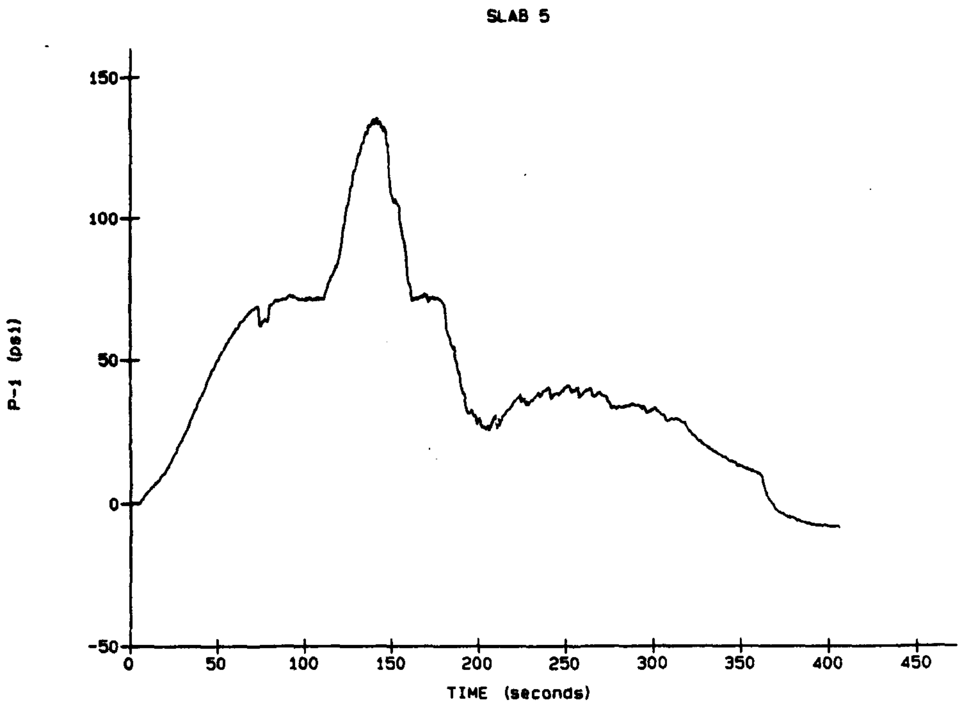


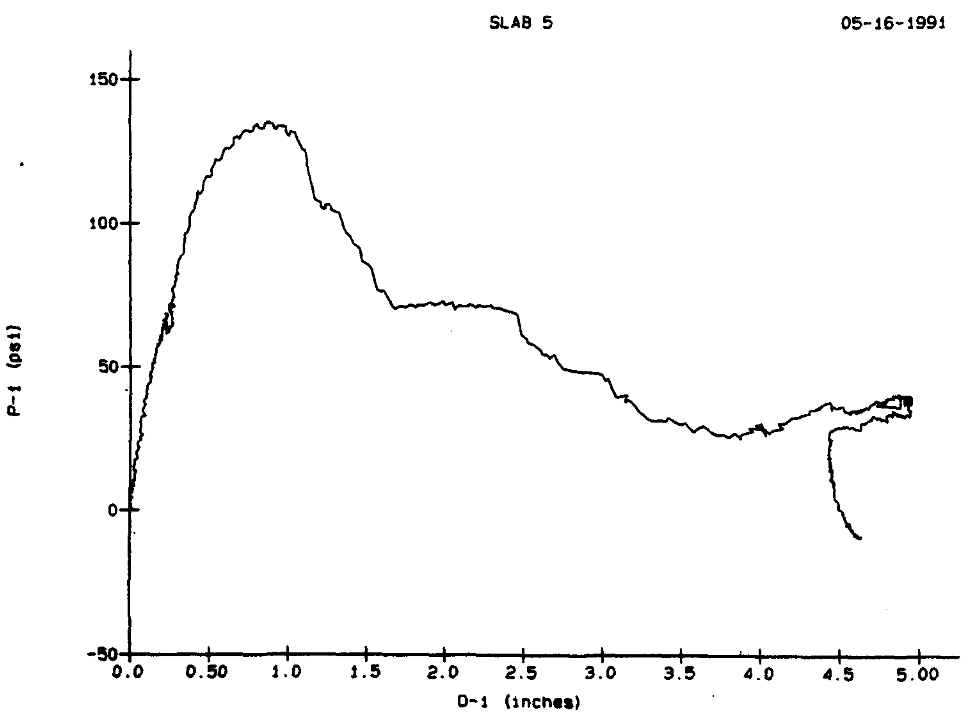
80
60
40
20
0
-20
P-1 (psi)
-1000
0
1000
2000
3000
4000
5000
6000
SL-5 (micro inch/inch)

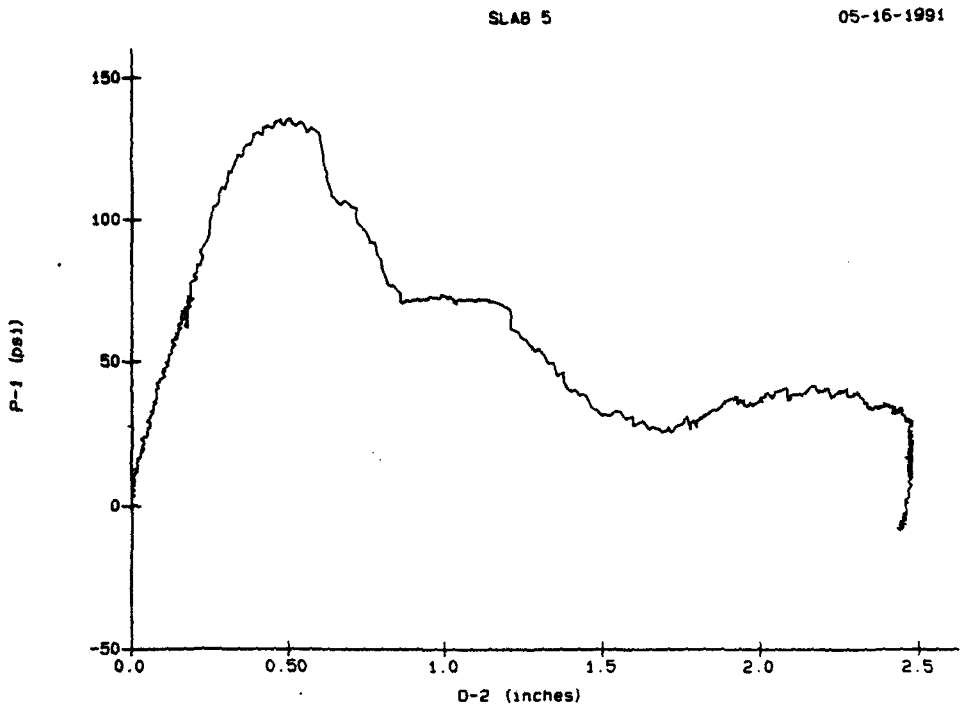
SLAB 4

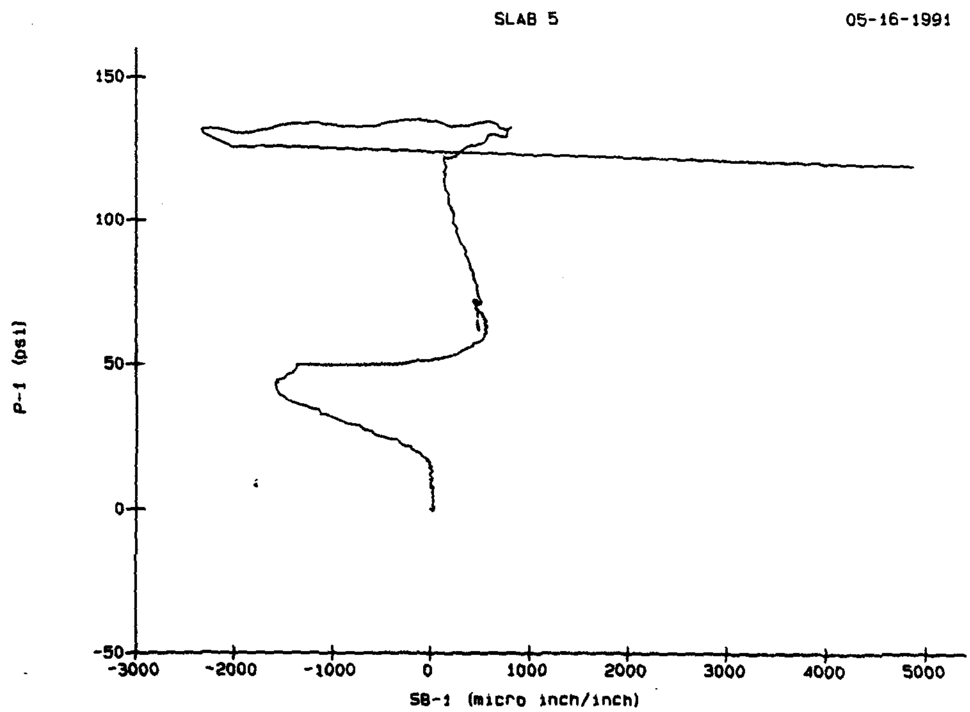


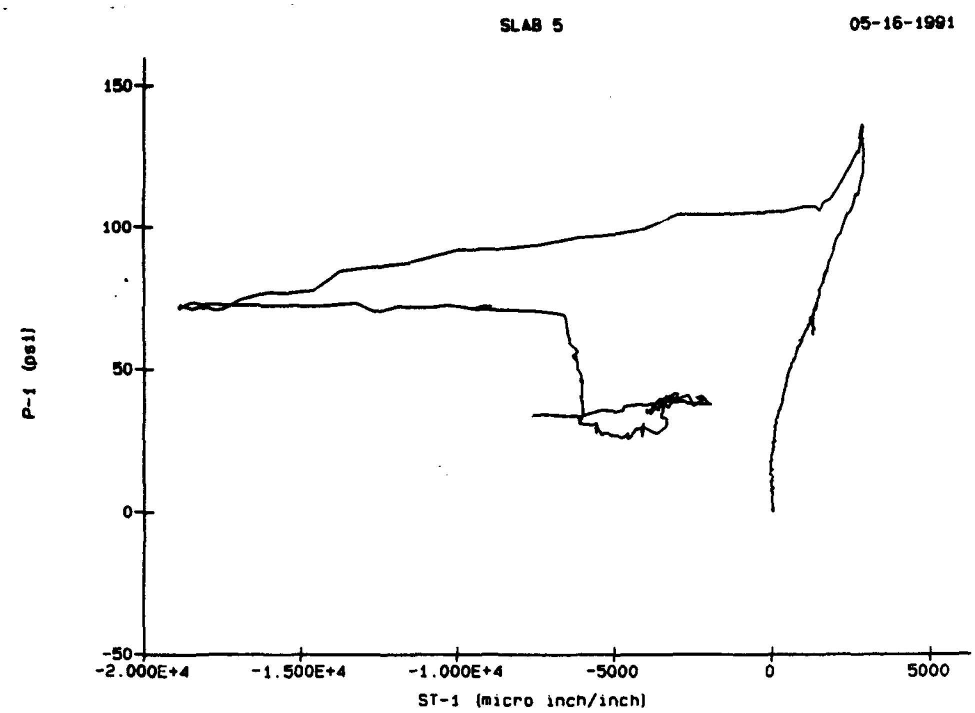
80
60
40
20
0
-20
P-1 (psi)
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
SL-6 (micro inch/inch)


SLAB 5
P-1 (psi)
TIME (seconds)
150
100
50
0
-50
0
50
100
150
200
250
300
350
400
450

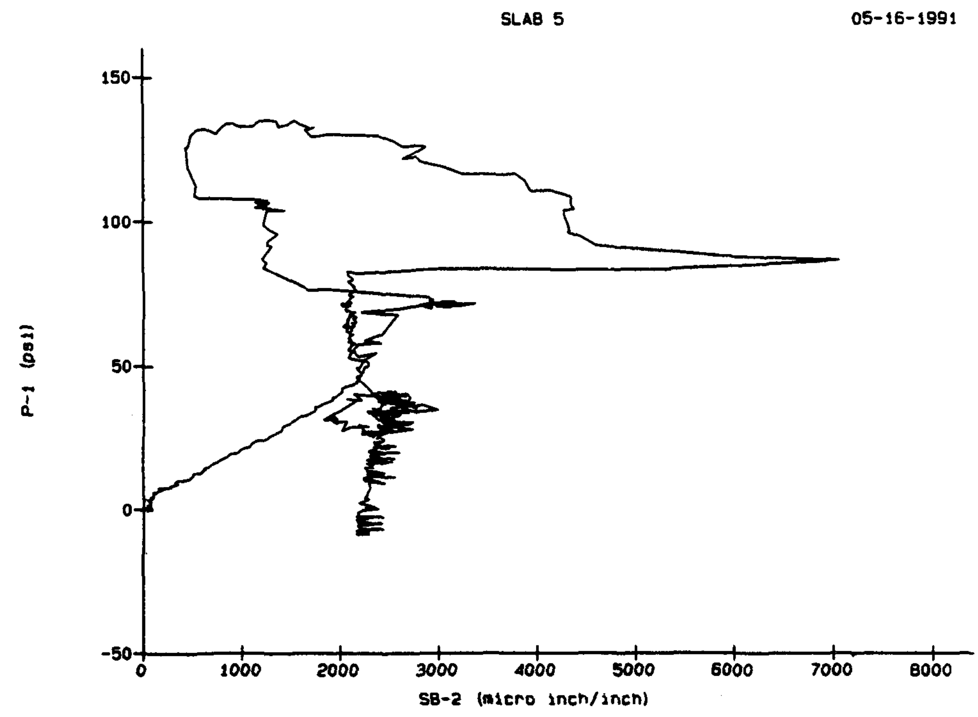

SLAB 5
05-16-1991
P-1 (psi)
D-1 (inches)
150
100
50
0
-50
0.0
0.50
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.00


SLAB 5
05-16-1991
150
100
50
0
-50
P-1 (psi)
0.0
0.50
1.0
1.5
2.0
2.5
D-2 (inches)

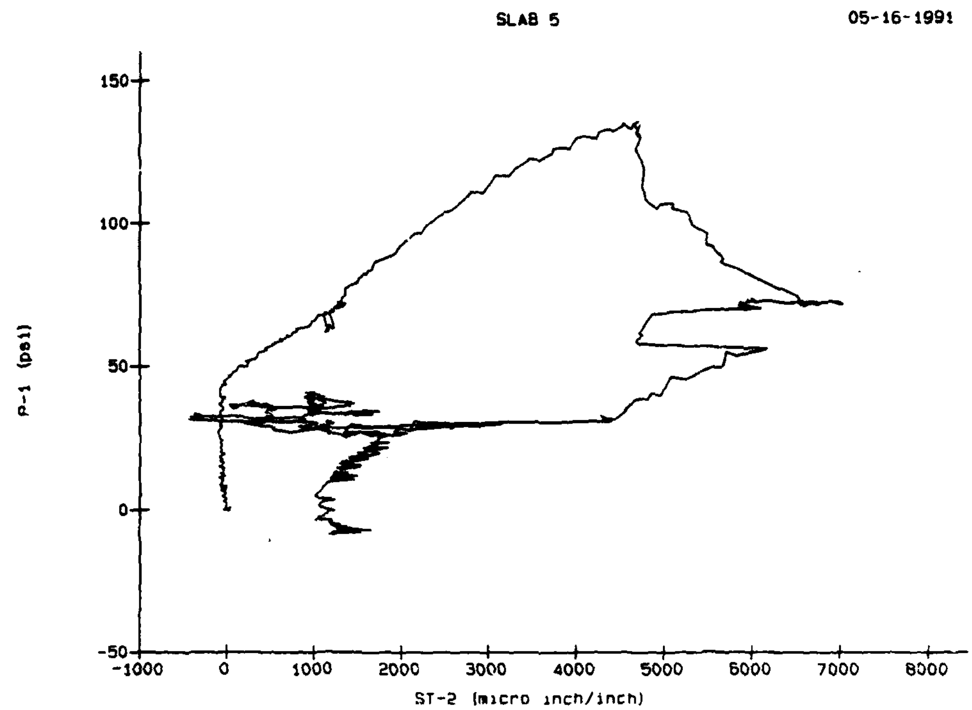

SLAB 5
05-16-1991
150
100
50
0
-50
P-1 (psi)
-3000
-2000
-1000
0
1000
2000
3000
4000
5000
SB-1 (micro inch/inch)

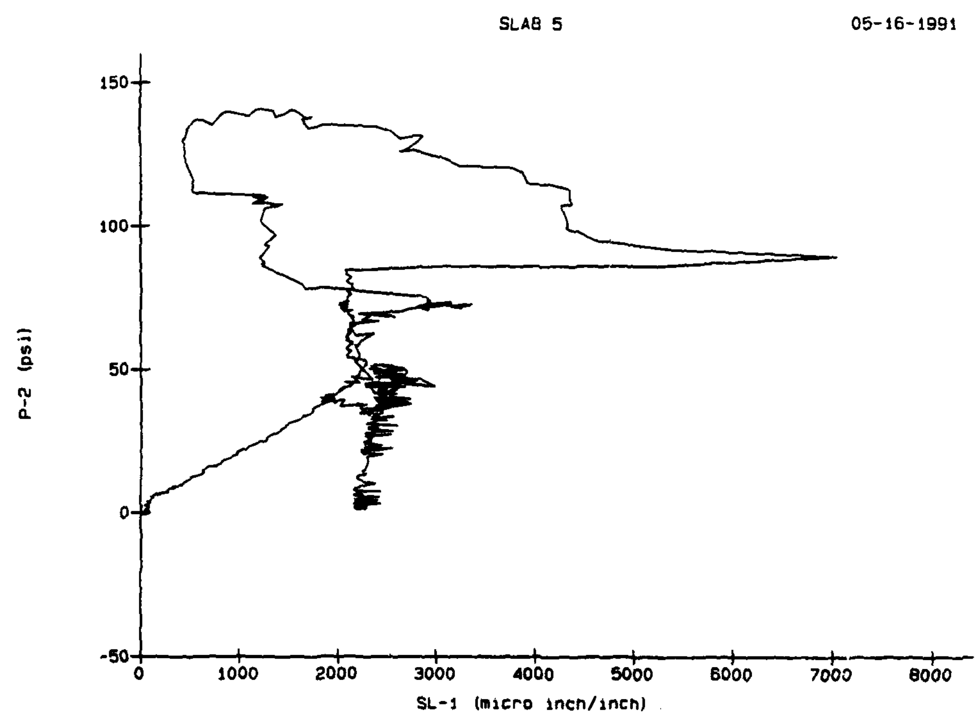


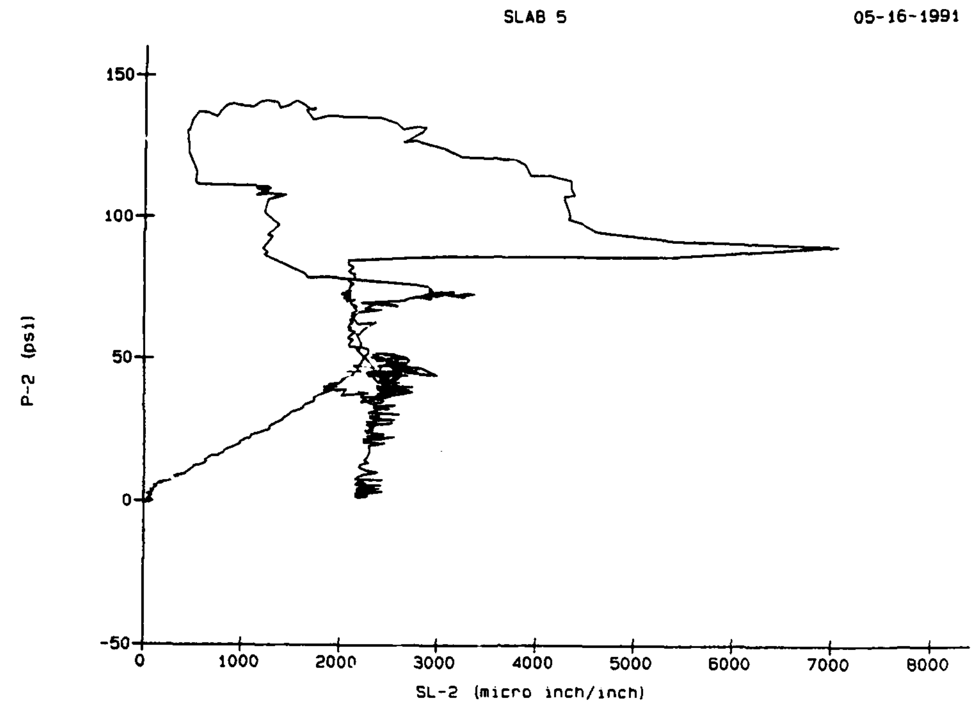
SLAB 5
05-16-1991
P-1 (psi)
150
100
50
0
-50
-2.000E+4
-1.500E+4
-1.000E+4
-5000
0
5000
ST-1 (micro inch/inch)

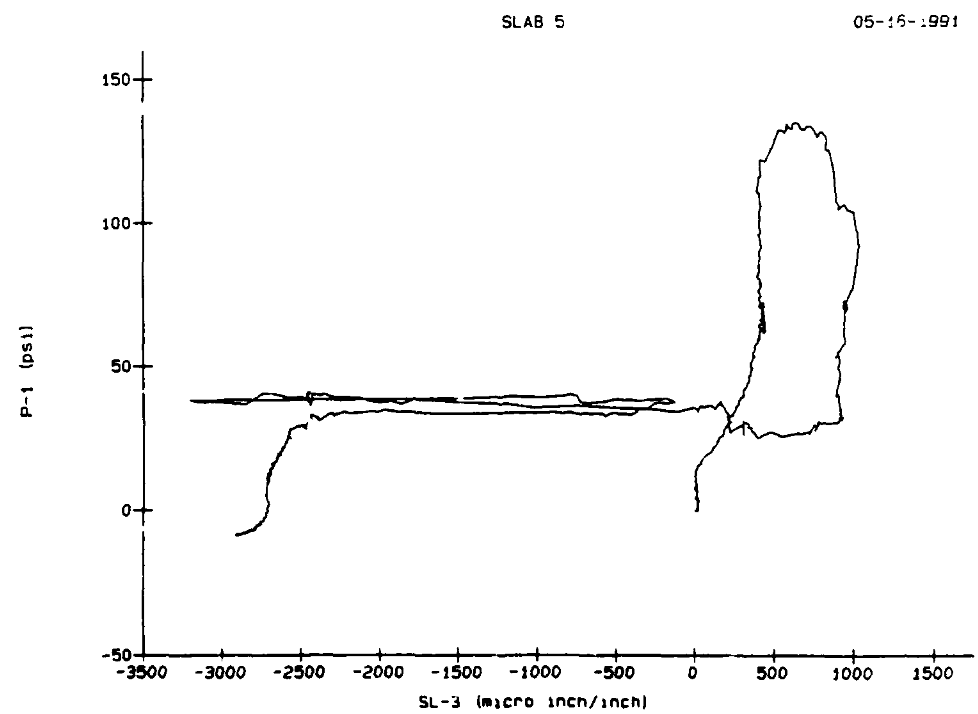


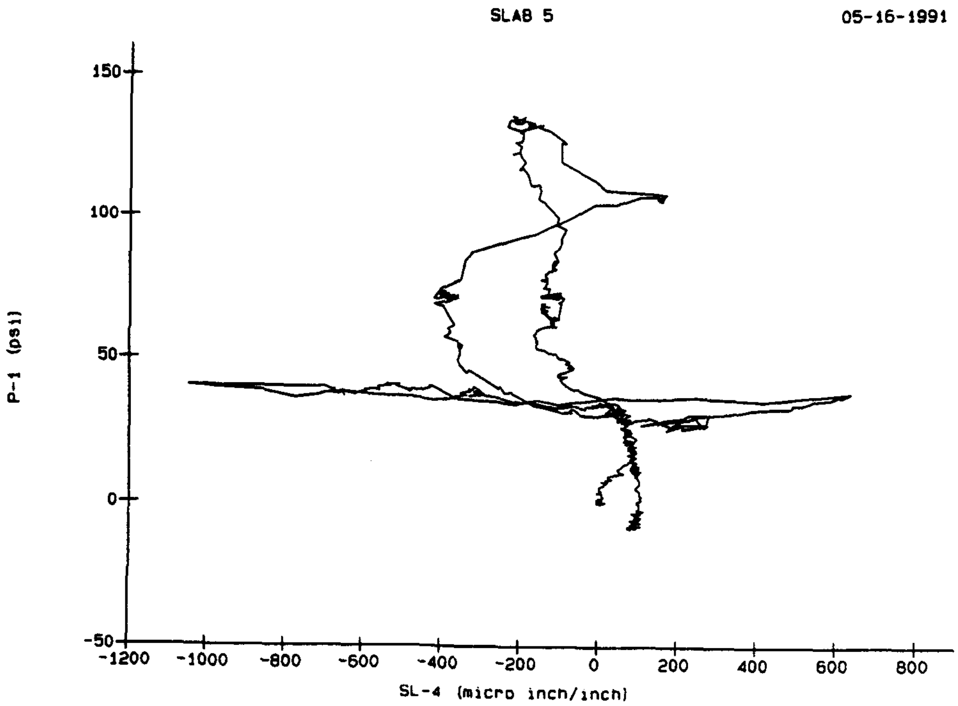
SLAB 5
05-16-1991
P-1 (psi)
150
100
50
0
-50
0
1000
2000
3000
4000
5000
6000
7000
8000
SB-2 (micro inch/inch)


SLAB 5
05-16-1991
150
100
50
0
-50
-1000
0
1000
2000
3000
4000
5000
6000
7000
8000
P-1 (psi)
ST-2 (micro inch/inch)

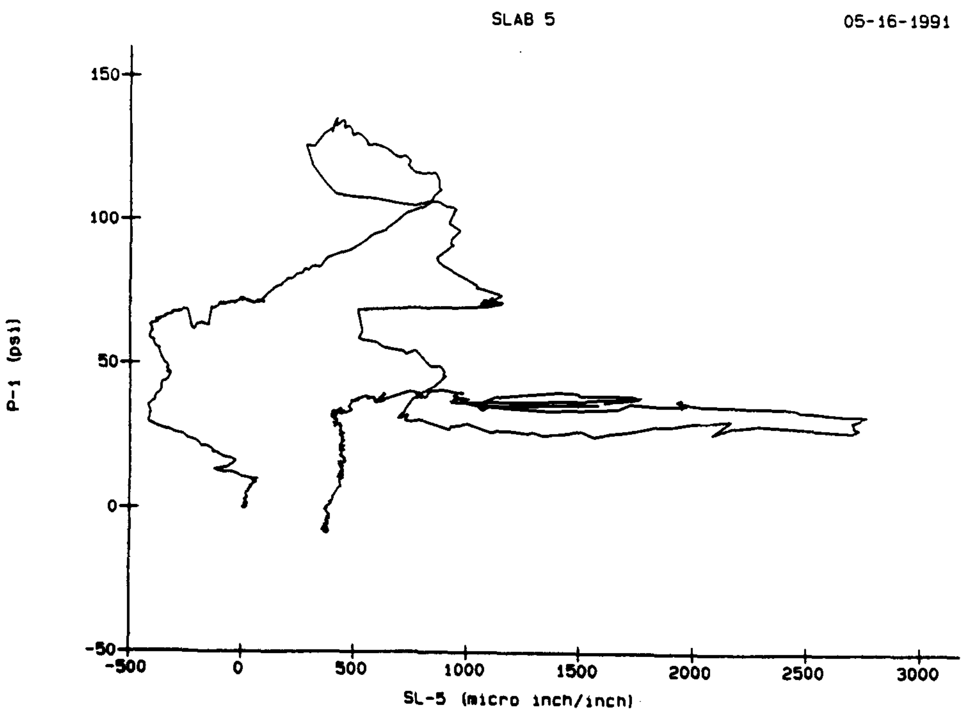

SLAB 5
05-16-1991
150
100
50
0
-50
0
1000
2000
3000
4000
5000
6000
7000
8000
P-2 (psi)
SL-1 (micro inch/inch)


SLAB 5
05-16-1991
P-2 (psi)
150
100
50
0
-50
0
1000
2000
3000
4000
5000
6000
7000
8000
SL-2 (micro inch/inch)


SLAB 5
P-1 (psi)
150
100
50
0
-50
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
SL-3 (micro inch/inch)

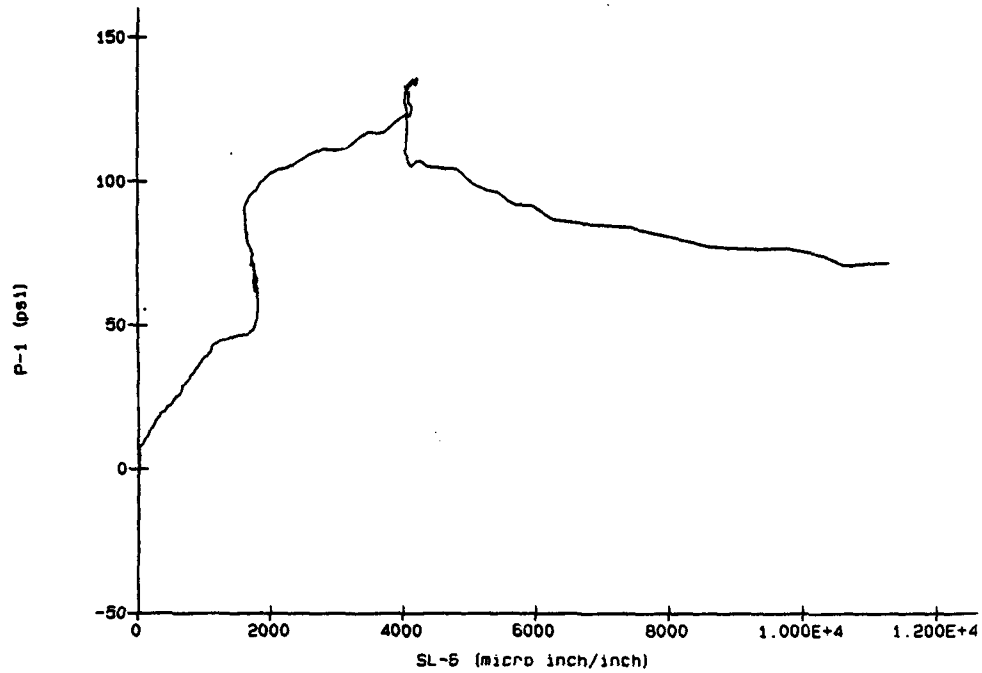


SLAB 5
05-16-1991
150
100
50
0
-50
-1200
-1000
-800
-600
-400
-200
0
200
400
600
800
SL-4 (micro inch/inch)
P-1 (psi)

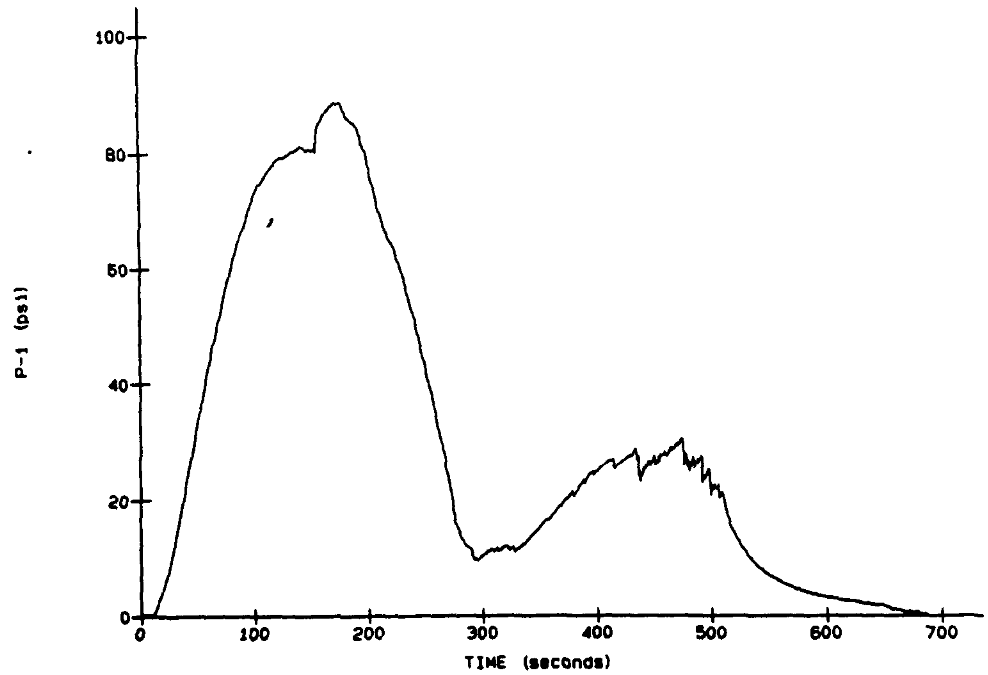


SLAB 5
05-16-1991
150
100
50
0
-50
-500
0
500
1000
1500
2000
2500
3000
SL-5 (micro inch/inch)
P-1 (psi)

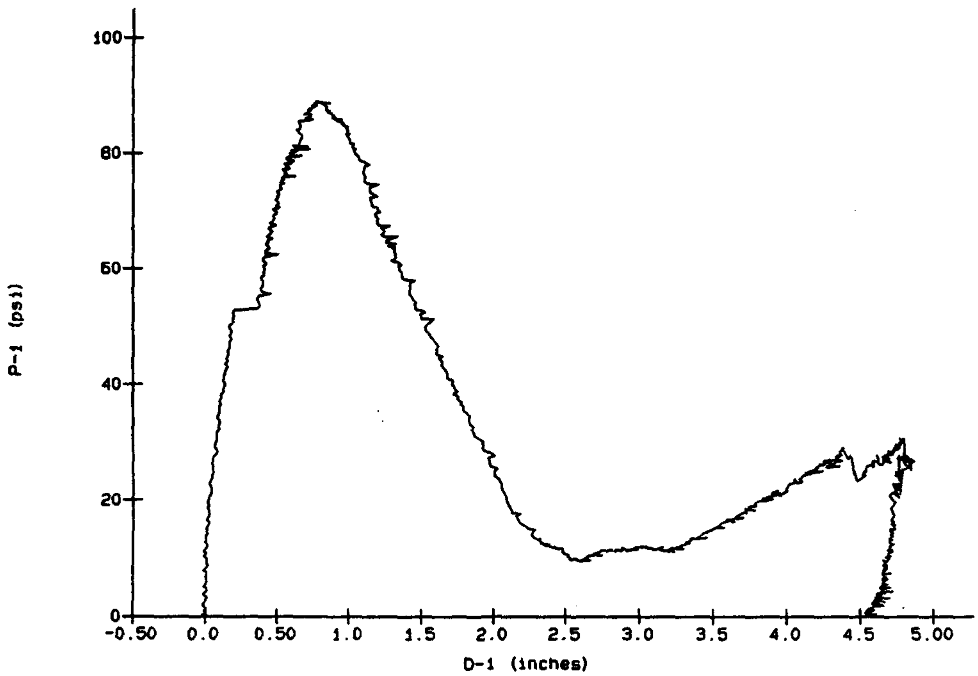
SLAB 5


P-1 (psi)
150
100
50
0
-50
0
2000
4000
6000
8000
1.000E+4
1.200E+4
SL-6 (micro inch/inch)

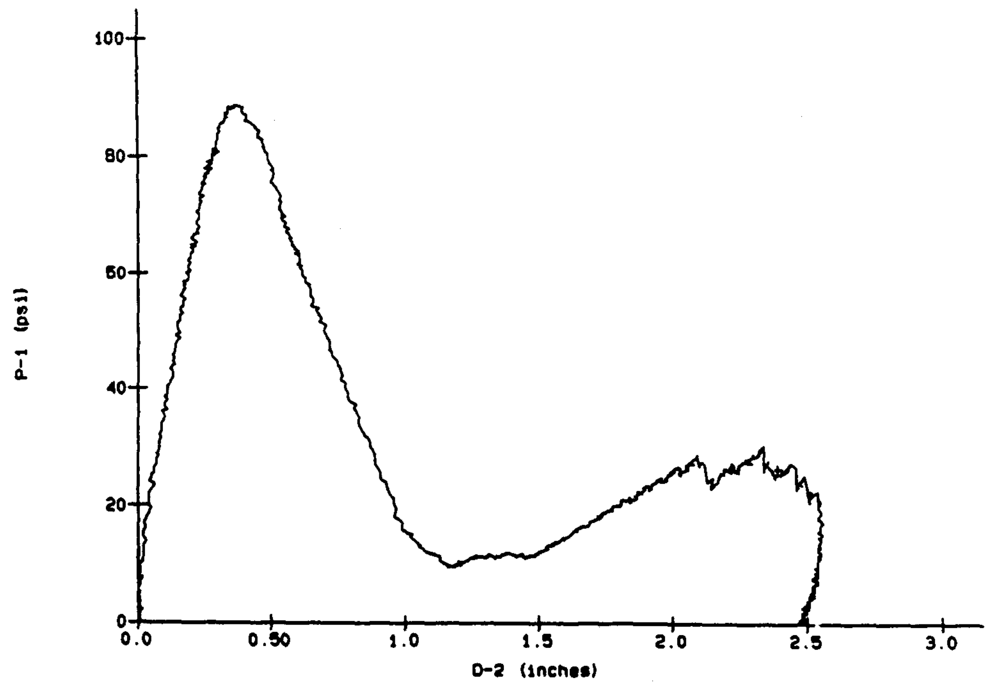
SLAB 6


P-1 (psi)
100
80
60
40
20
0
0
100
200
300
400
500
600
700
TIME (seconds)

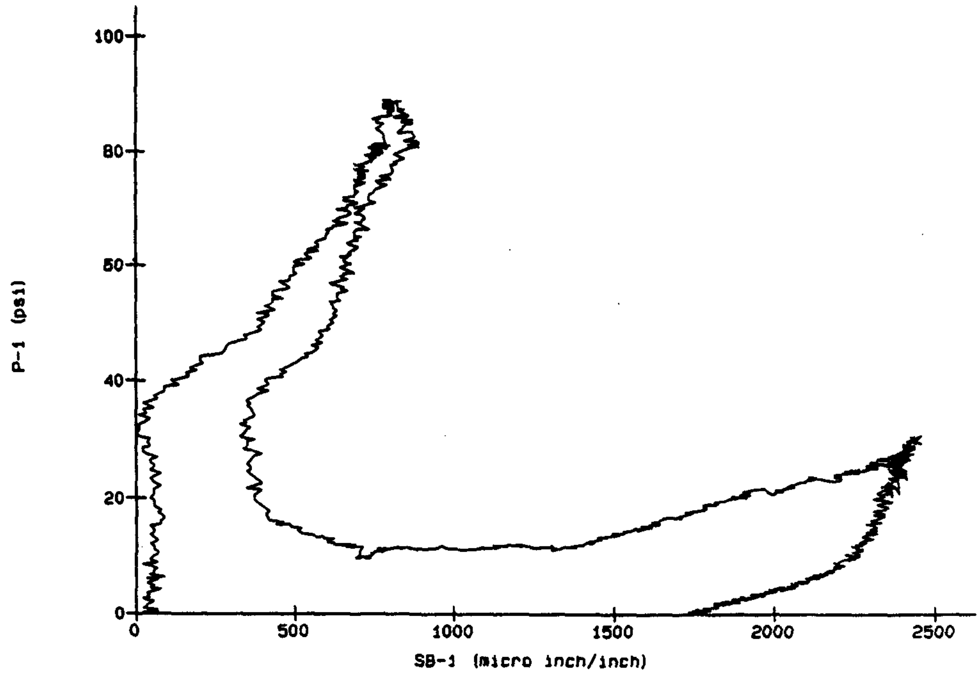
SLAB 6


P-1 (psi)
D-1 (inches)
100
80
60
40
20
0
-0.50
0.0
0.50
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.00

SLAB 6

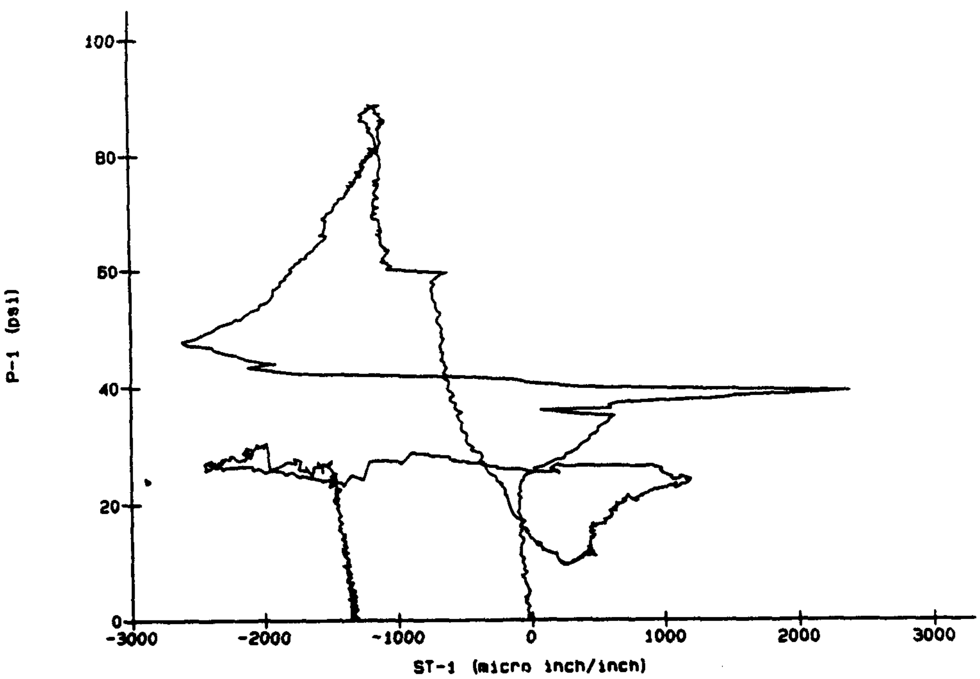

P-1 (psi)
D-2 (inches)
100
80
60
40
20
0
0.0
0.50
1.0
1.5
2.0
2.5
3.0

SLAB 6

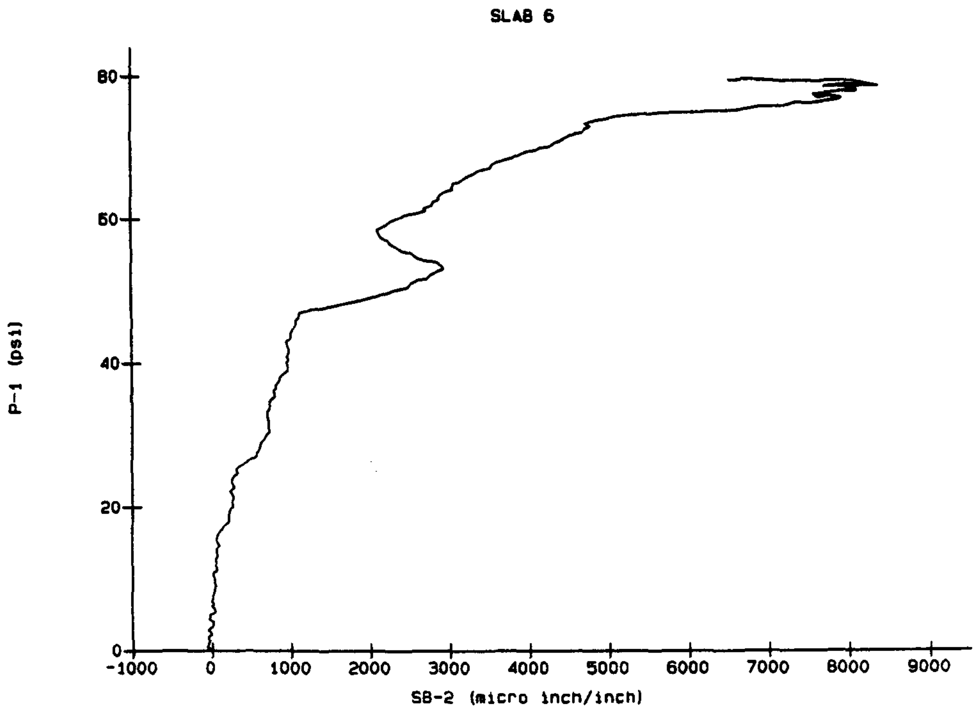


100
80
60
40
20
0
P-1 (psi)
0
500
1000
1500
2000
2500
SB-1 (micro inch/inch)

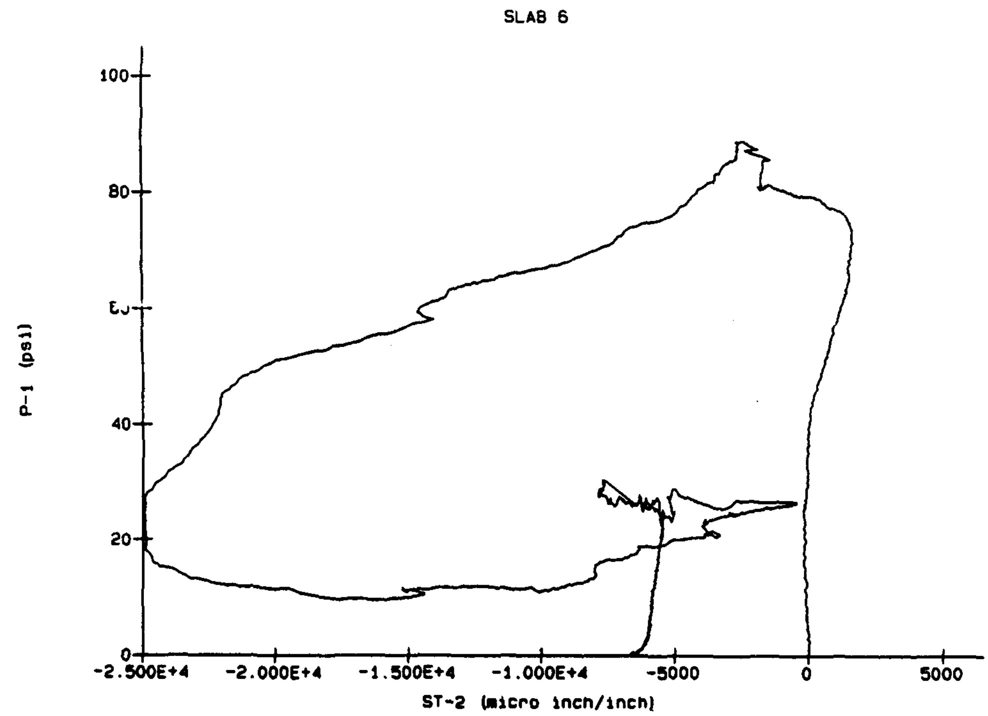
SLAB 6



100
80
60
40
20
0
P-1 (psi)
-3000
-2000
-1000
0
1000
2000
3000
ST-1 (micro inch/inch)

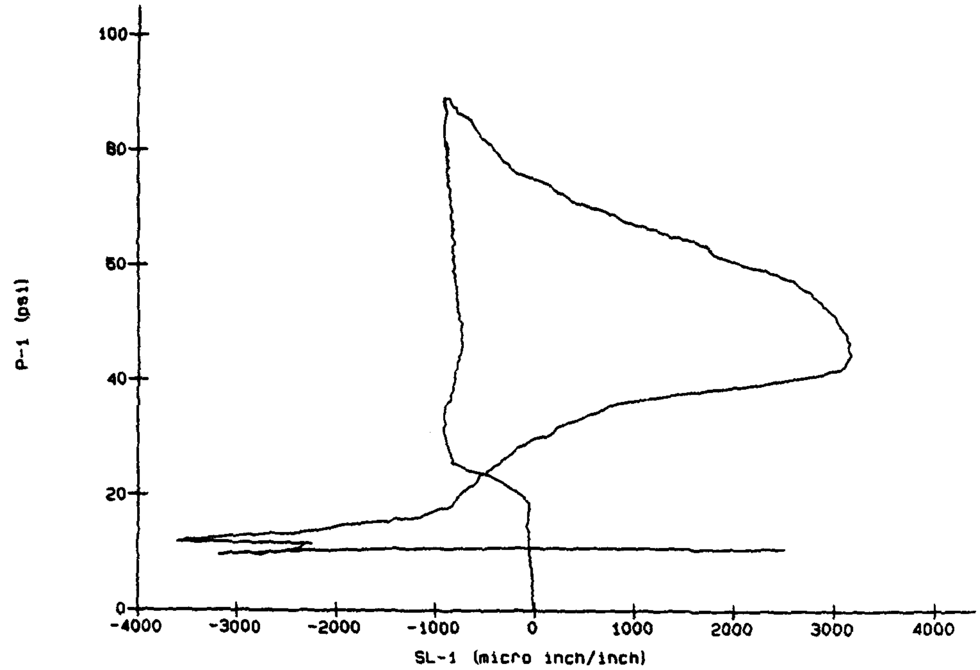


SLAB 6
P-1 (psi)
SB-2 (micro inch/inch)

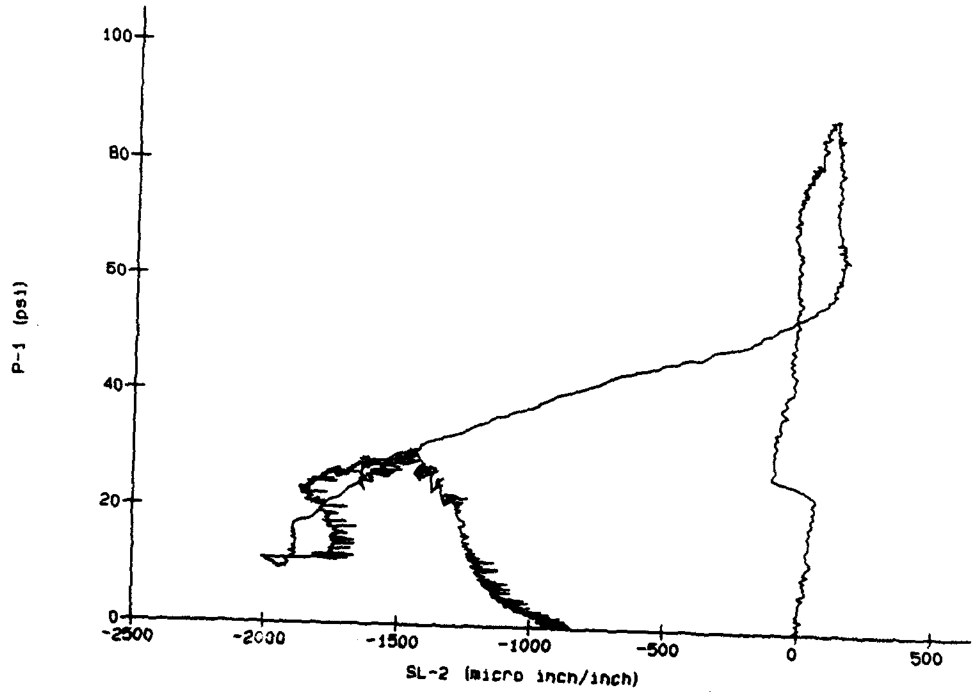


SLAB 6
P-1 (psi)
ST-2 (micro inch/inch)

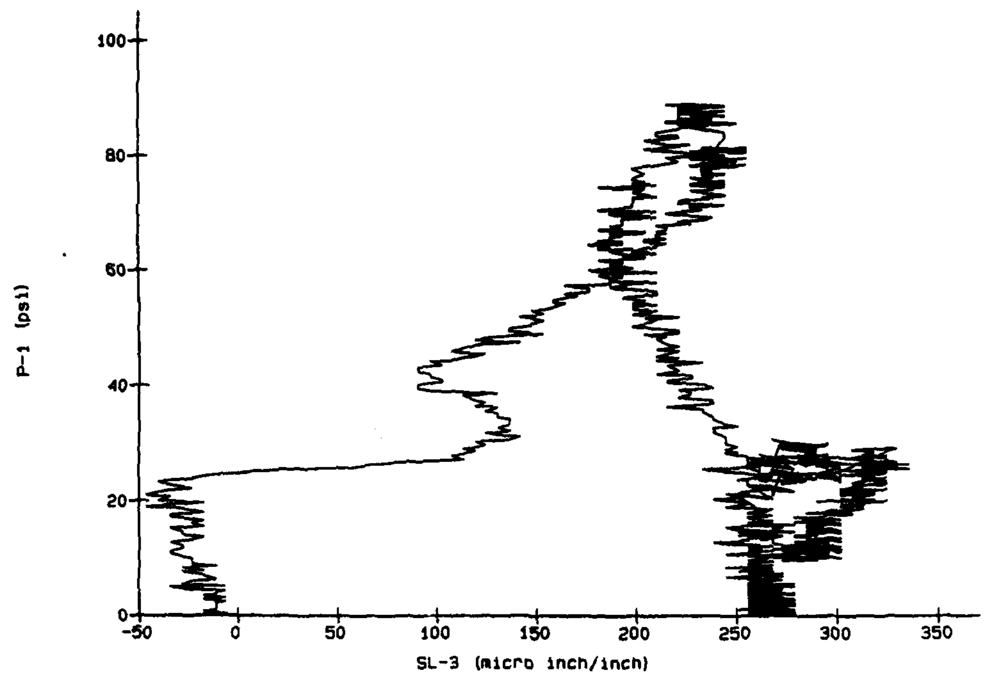
SLAB 6


100
80
60
40
20
0
P-1 (psi)
-4000
-3000
-2000
-1000
0
1000
2000
3000
4000
SL-1 (micro inch/inch)

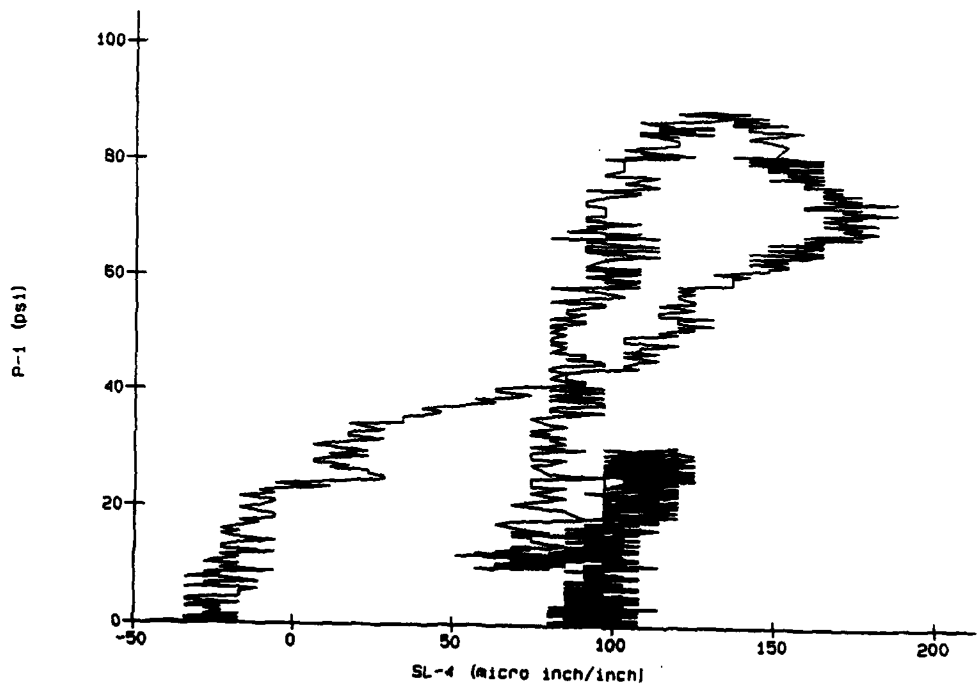
SLAB 6


100
80
60
40
20
0
P-1 (psi)
-2500
-2000
-1500
-1000
-500
0
500
SL-2 (micro inch/inch)

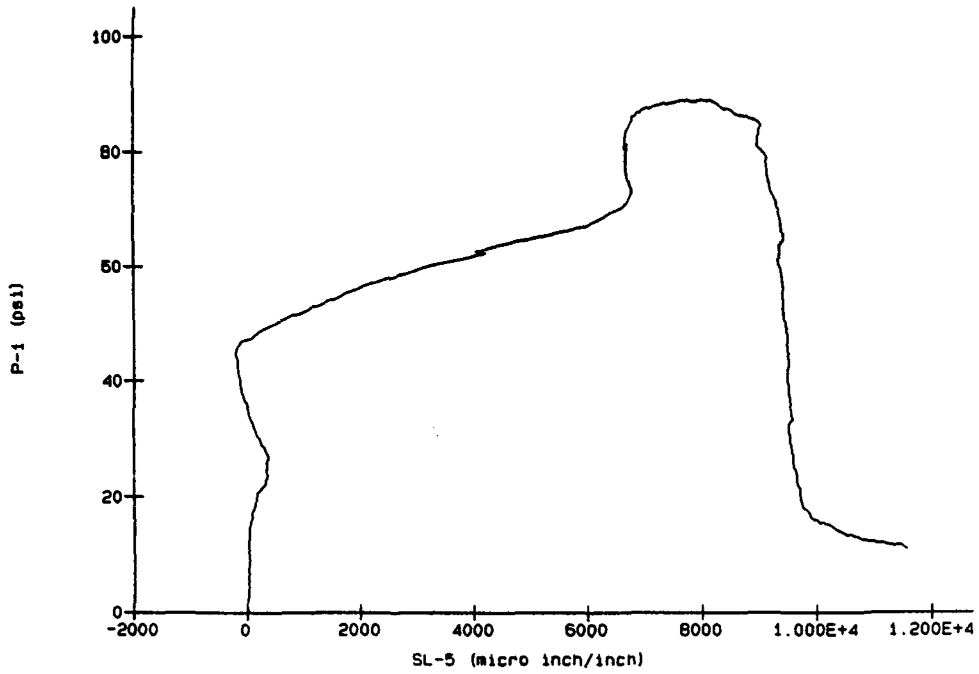
SLAB 6


100
80
60
40
20
0
P-1 (psi)
-50
0
50
100
150
200
250
300
350
SL-3 (micro inch/inch)

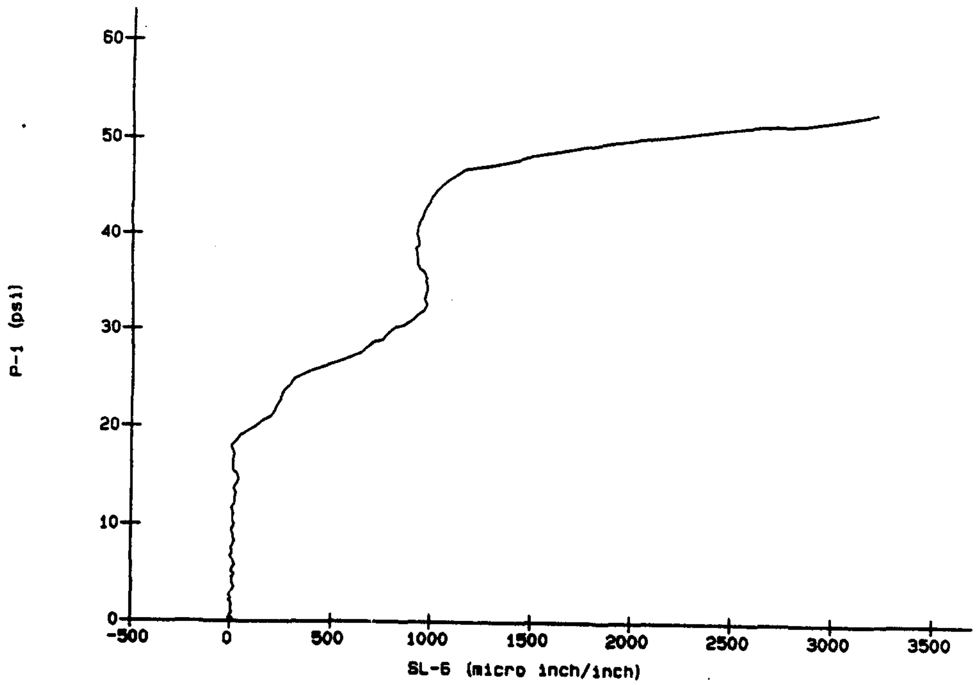
SLAB 6

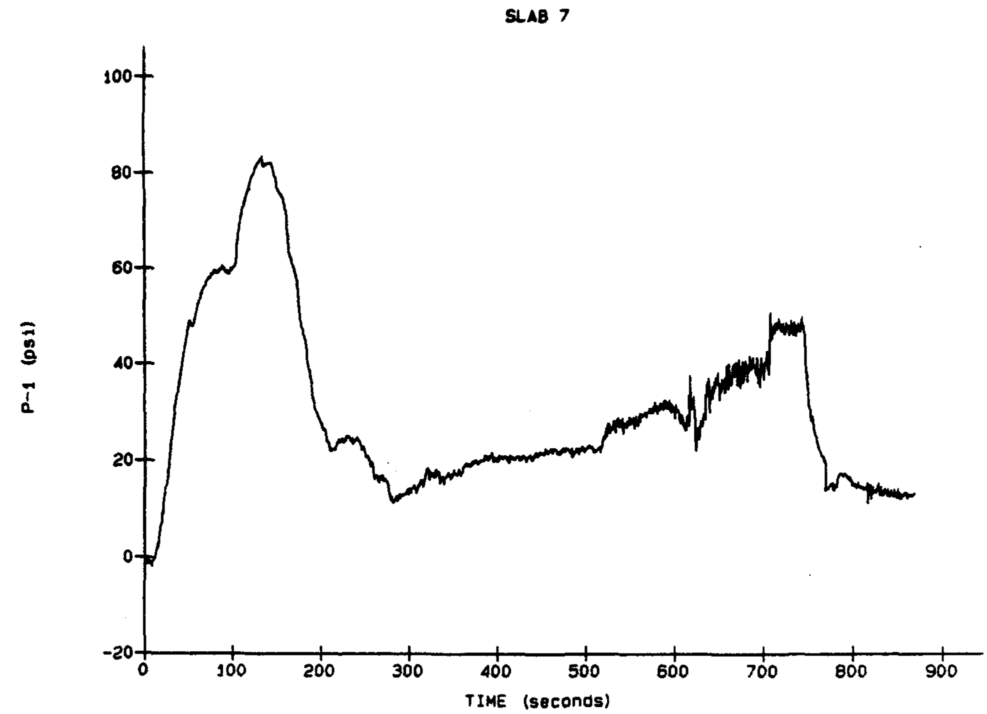

100
80
60
40
20
0
P-1 (psi)
-50
0
50
100
150
200
SL-4 (micro inch/inch)

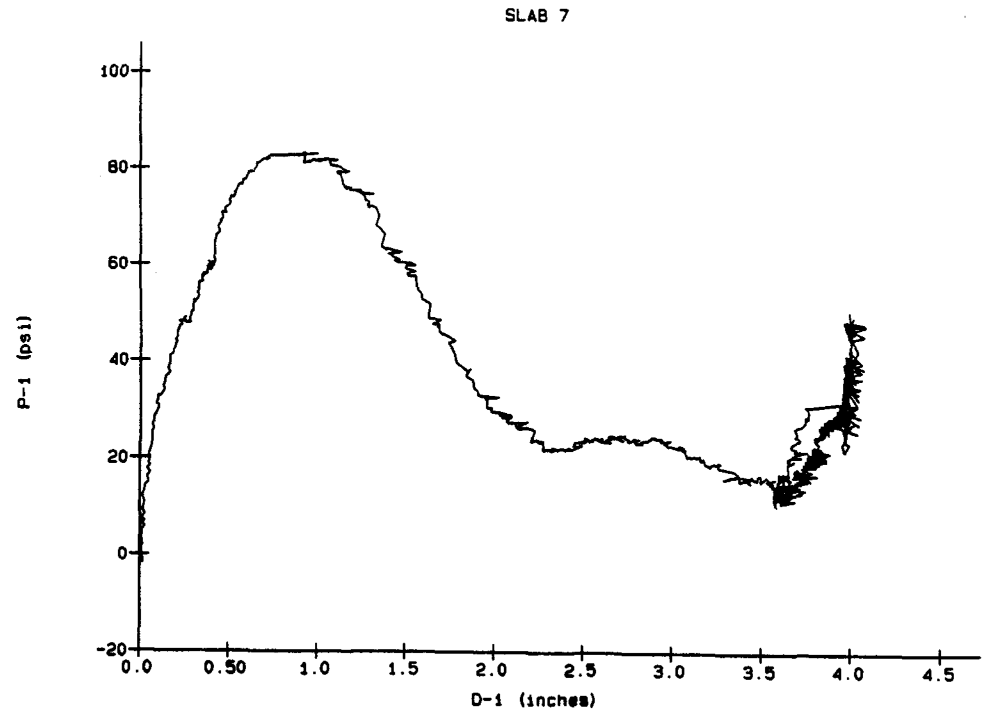
SLAB 6


100
80
60
40
20
0
-2000
0
2000
4000
6000
8000
1.000E+4
1.200E+4
SL-5 (micro inch/inch)
P-1 (psi)

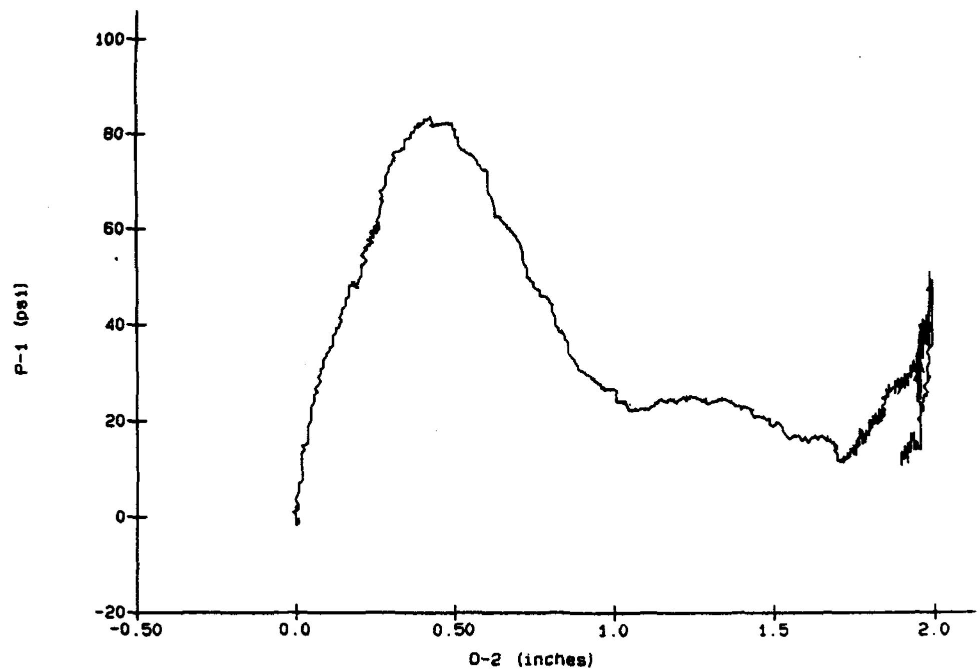
SLAB 6


60
50
40
30
20
10
0
-500
0
500
1000
1500
2000
2500
3000
3500
SL-6 (micro inch/inch)
P-1 (psi)

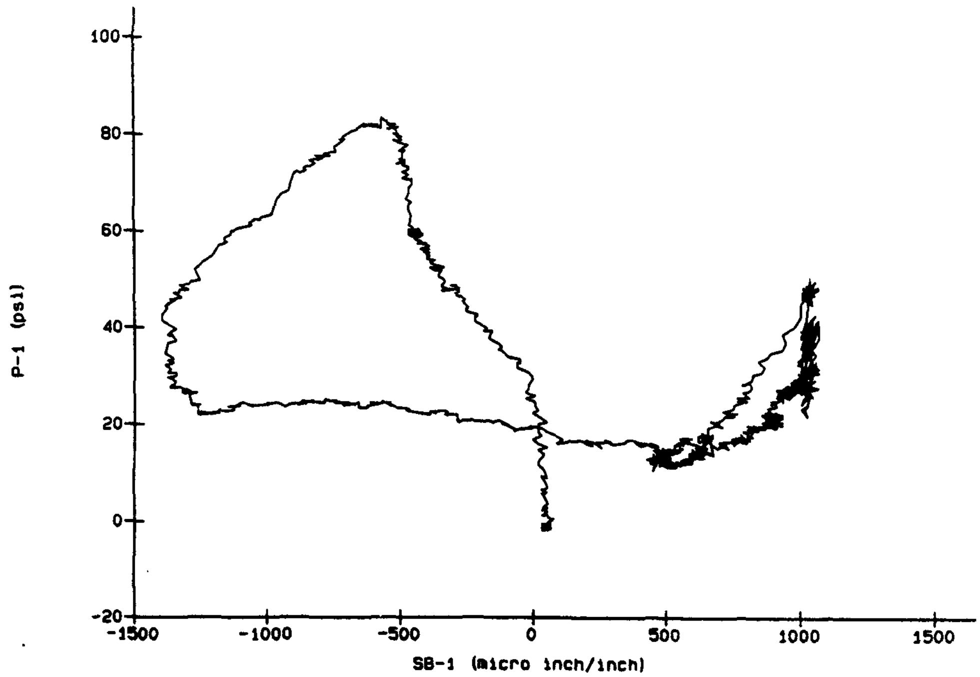

SLAB 7
P-1 (psi)
100
80
60
40
20
0
-20
0
100
200
300
400
500
600
700
800
900
TIME (seconds)

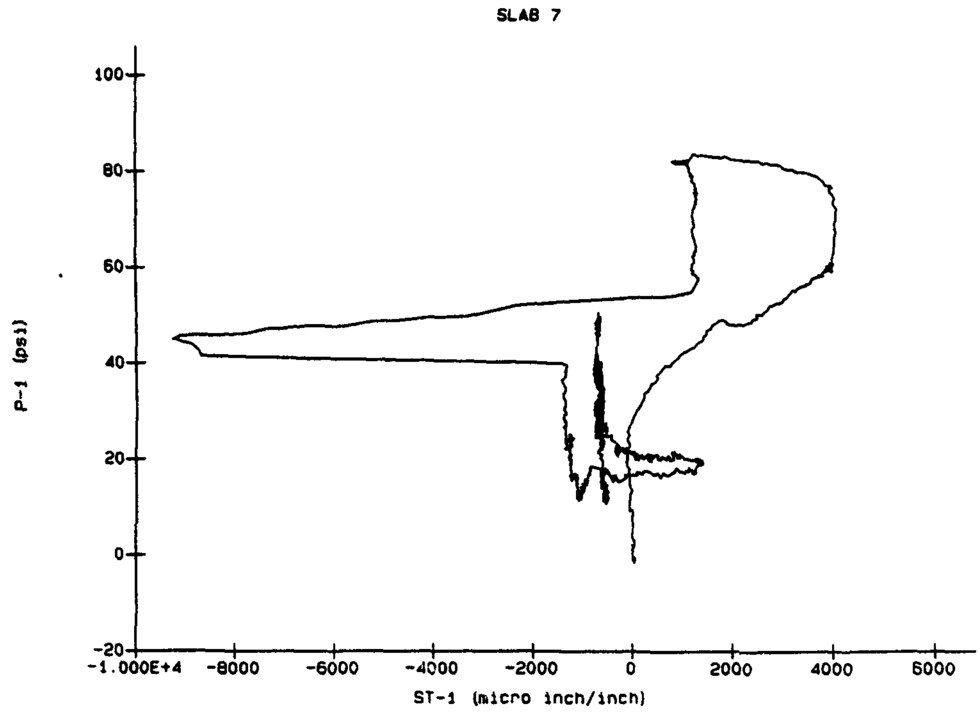

SLAB 7
P-1 (psi)
100
80
60
40
20
0
-20
0.0
0.50
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
D-1 (inches)

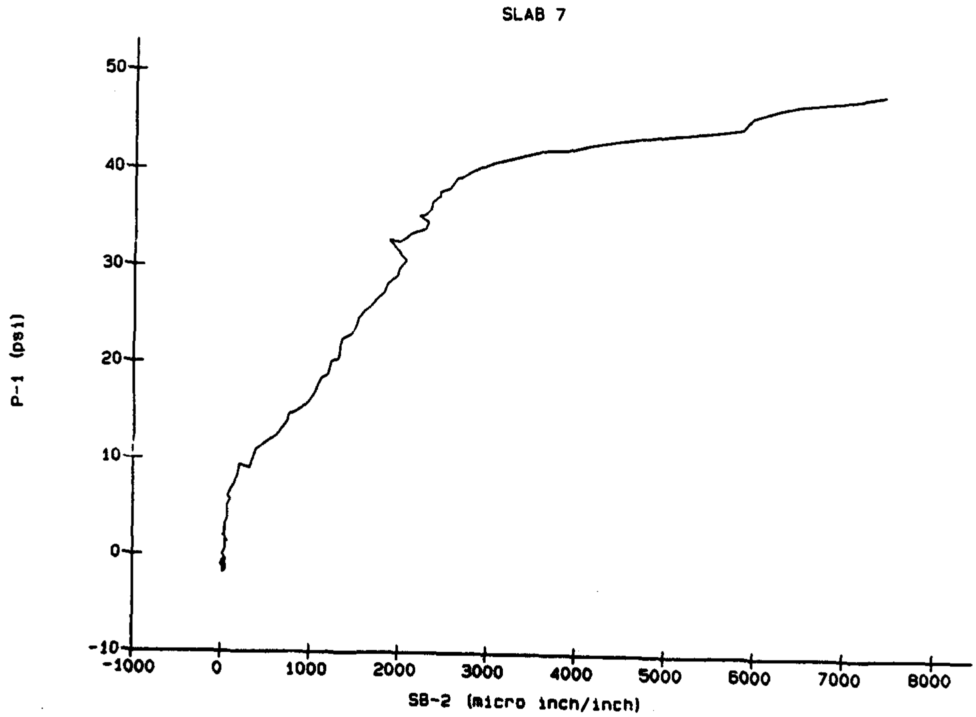
SLAB 7


100
80
60
40
20
0
-20
P-1 (psi)
-0.50
0.0
0.50
1.0
1.5
2.0
D-2 (inches)

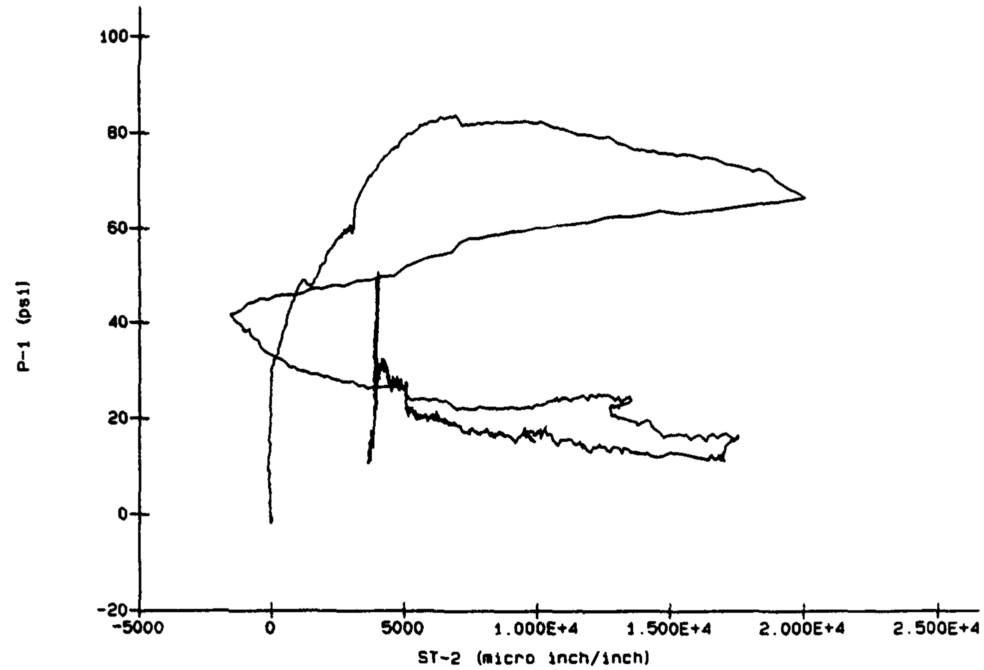
SLAB 7


100
80
60
40
20
0
-20
P-1 (psi)
-1500
-1000
-500
0
500
1000
1500
SB-1 (micro inch/inch)

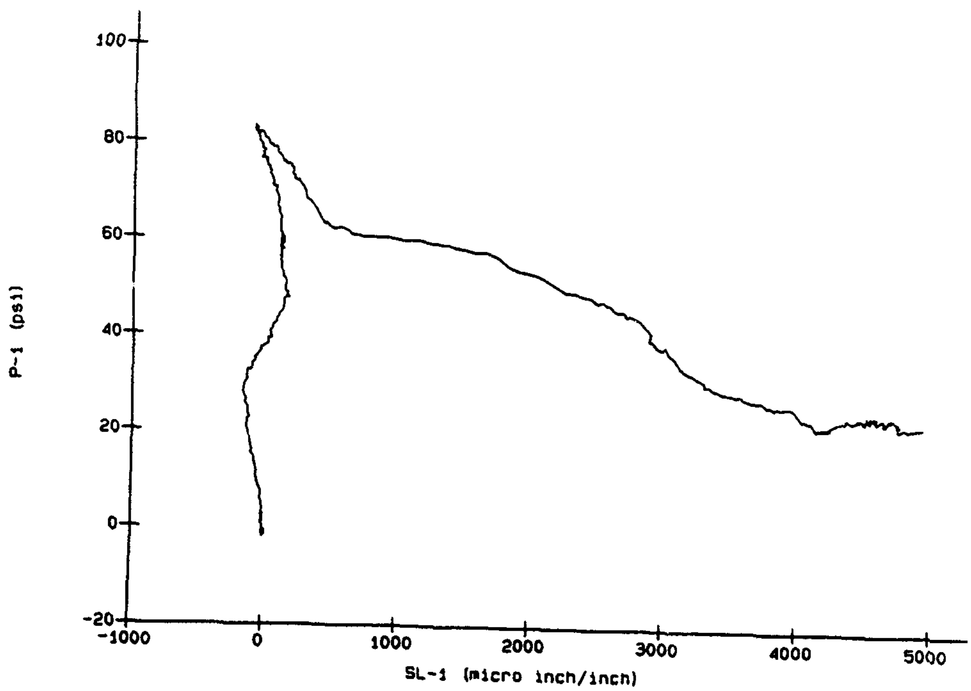

SLAB 7
P-1 (psi)
100
80
60
40
20
0
-20
-1.000E+4
-8000
-6000
-4000
-2000
0
2000
4000
6000
ST-1 (micro inch/inch)

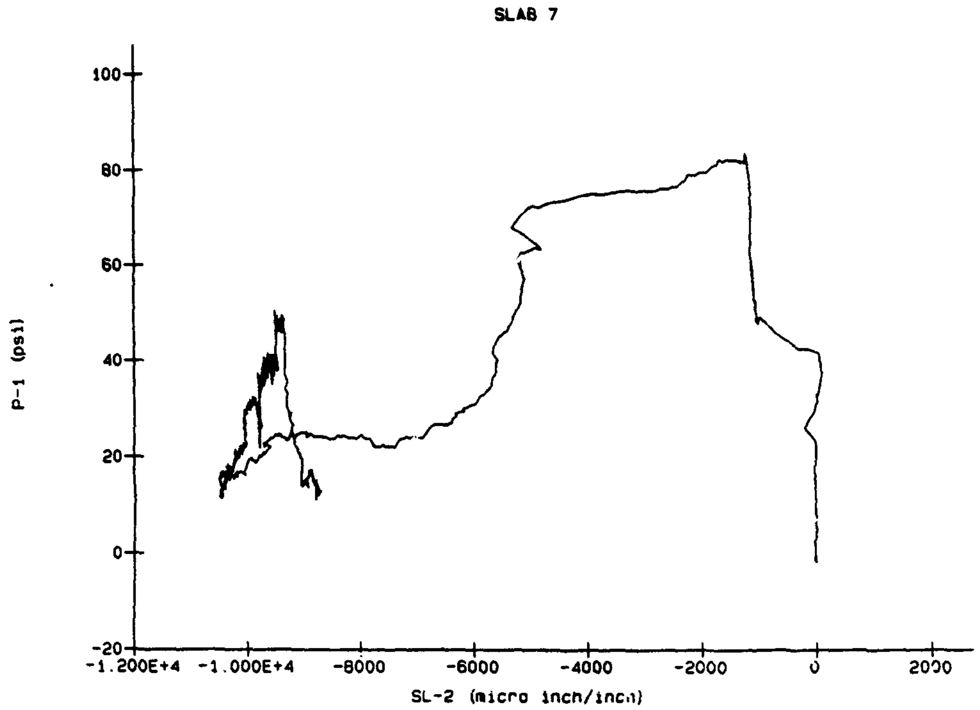

SLAB 7
P-1 (psi)
50
40
30
20
10
0
-10
-1000
0
1000
2000
3000
4000
5000
6000
7000
8000
SB-2 (micro inch/inch)

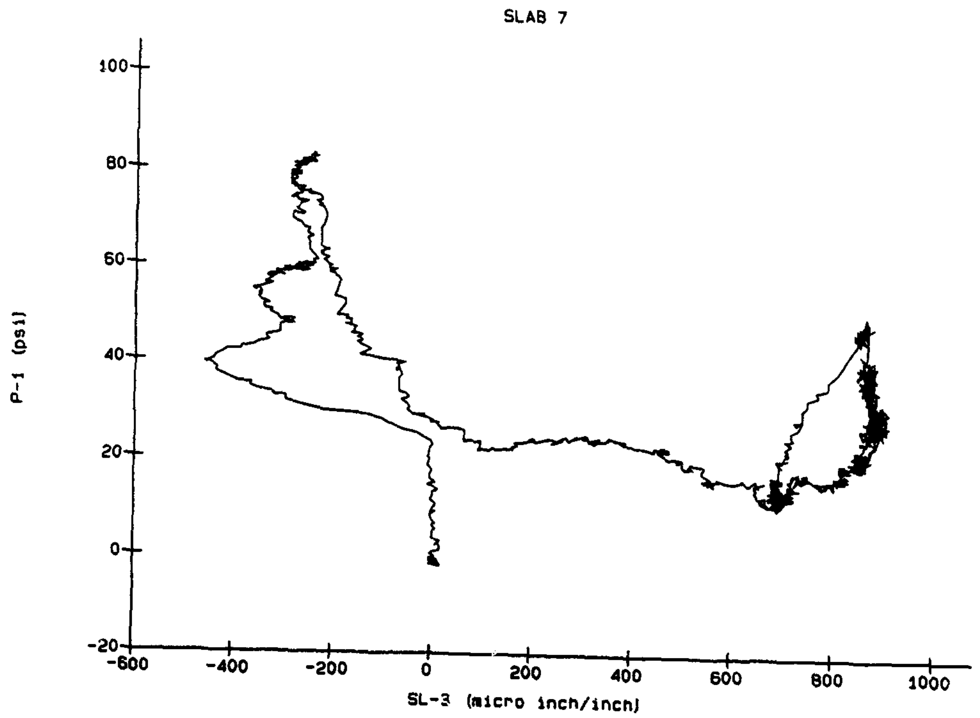
SLAB 7


100
80
60
40
20
0
-20
P-1 (psi)
-5000
0
5000
1.000E+4
1.500E+4
2.000E+4
2.500E+4
ST-2 (micro inch/inch)

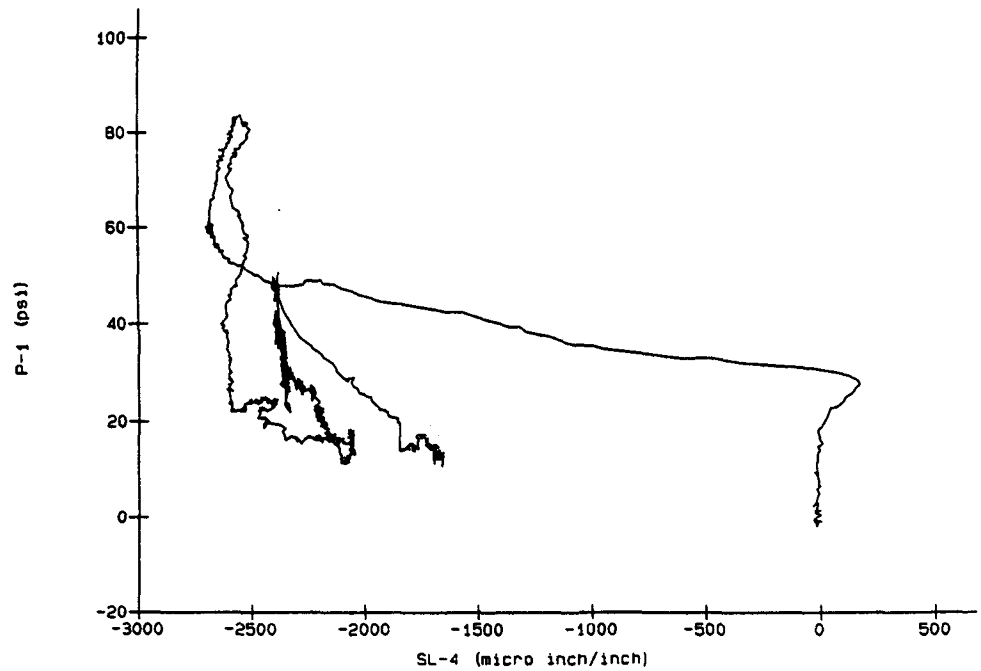
SLAB 7


100
80
60
40
20
0
-20
P-1 (psi)
-1000
0
1000
2000
3000
4000
5000
SL-1 (micro inch/inch)


SLAB 7
P-1 (psi)
100
80
60
40
20
0
-20
-1.200E+4
-1.000E+4
-8000
-6000
-4000
-2000
0
2000
SL-2 (micro inch/inch)

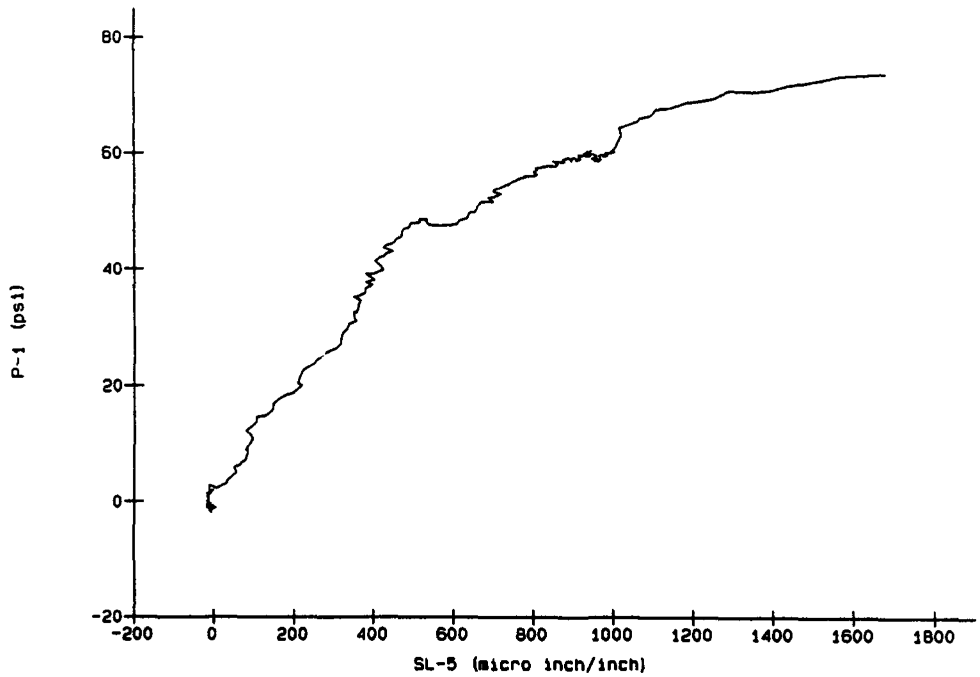

SLAB 7
P-1 (psi)
100
80
60
40
20
0
-20
-600
-400
-200
0
200
400
600
800
1000
SL-3 (micro inch/inch)

SLAB 7

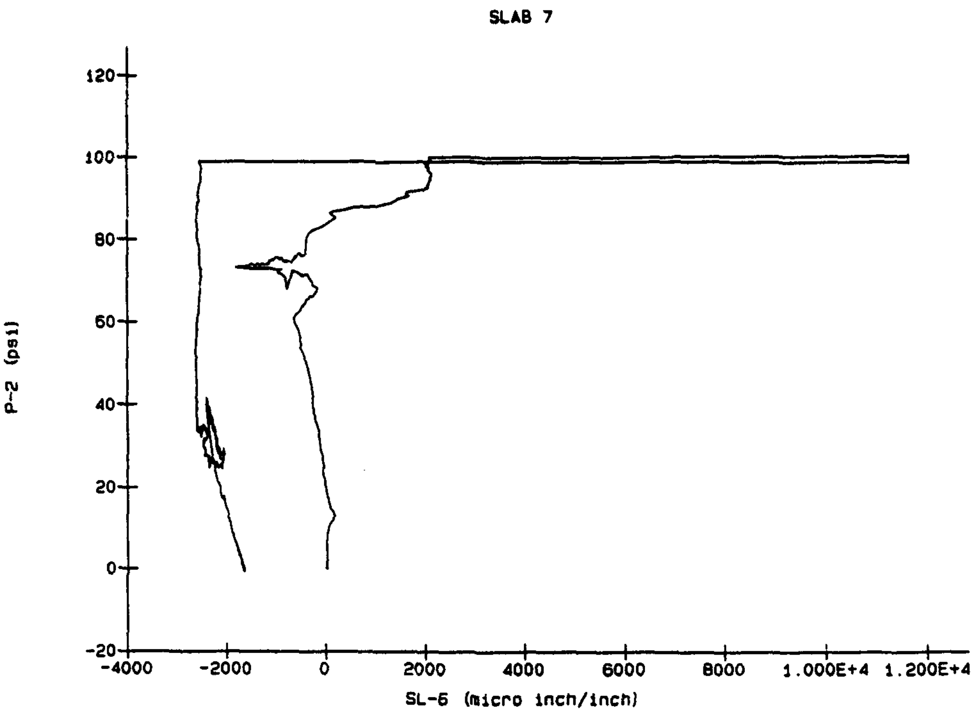


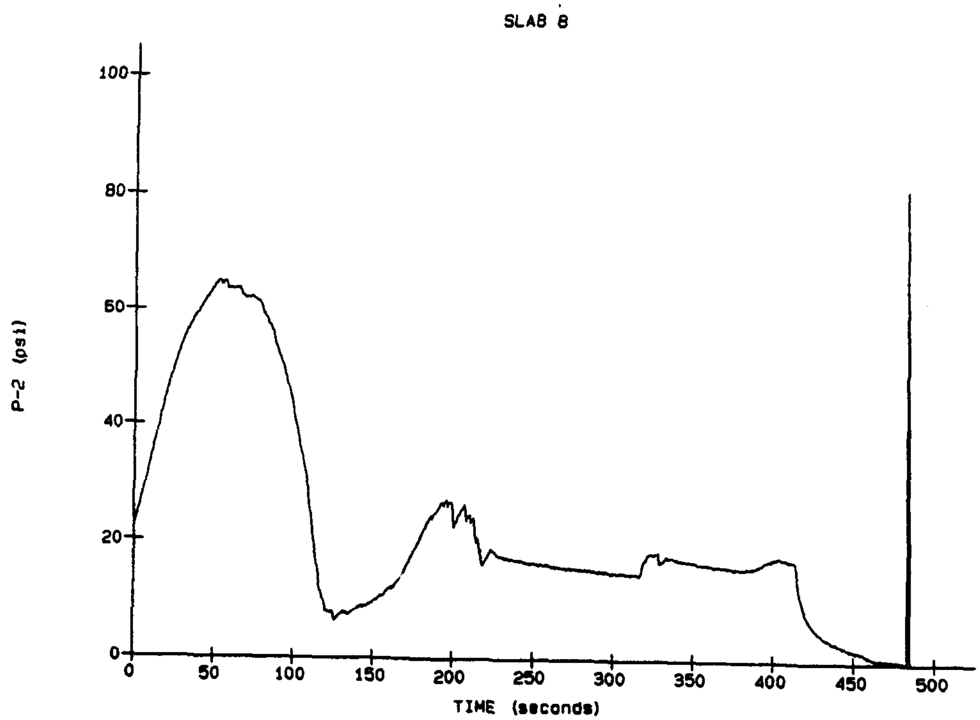
P-1 (psi)
100
80
60
40
20
0
-20
-3000
-2500
-2000
-1500
-1000
-500
0
500
SL-4 (micro inch/inch)

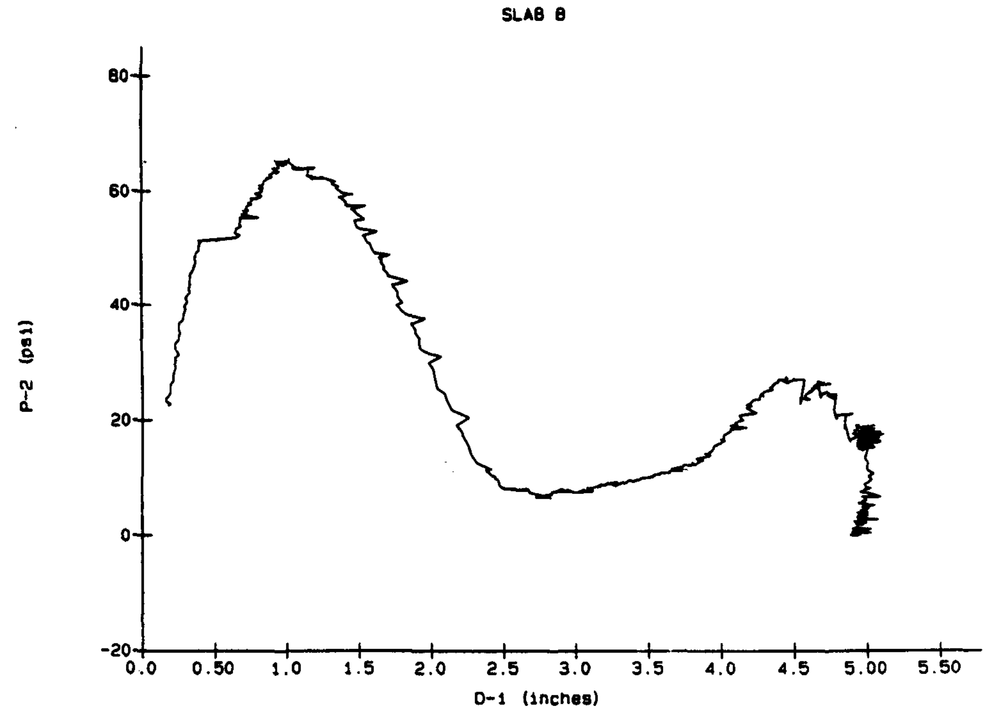
SLAB 7

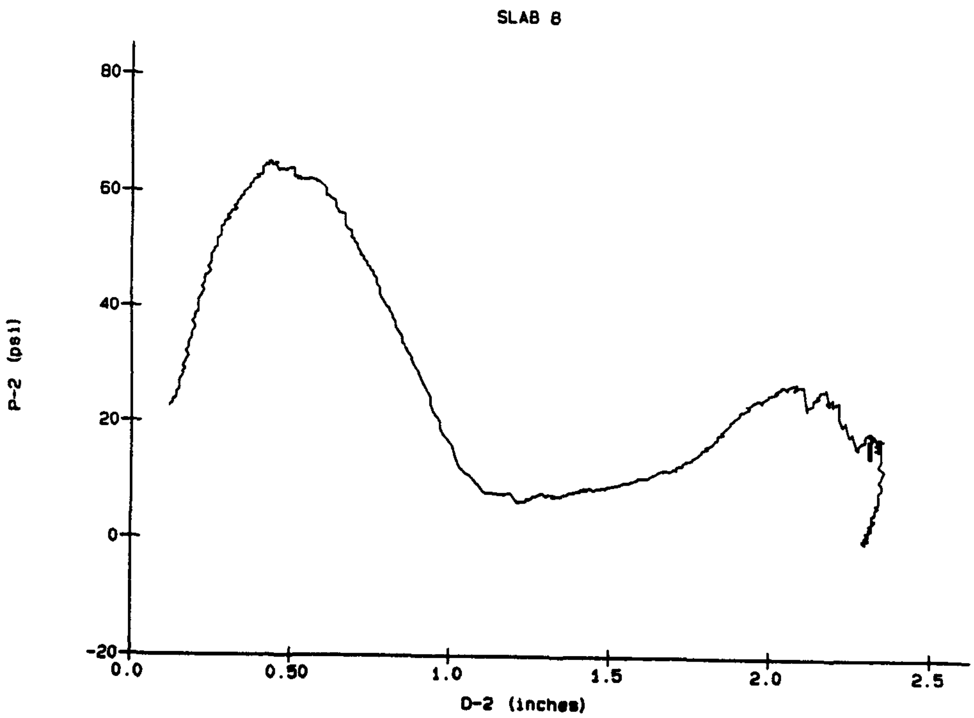


P-1 (psi)
80
60
40
20
0
-20
-200
0
200
400
600
800
1000
1200
1400
1600
1800
SL-5 (micro inch/inch)

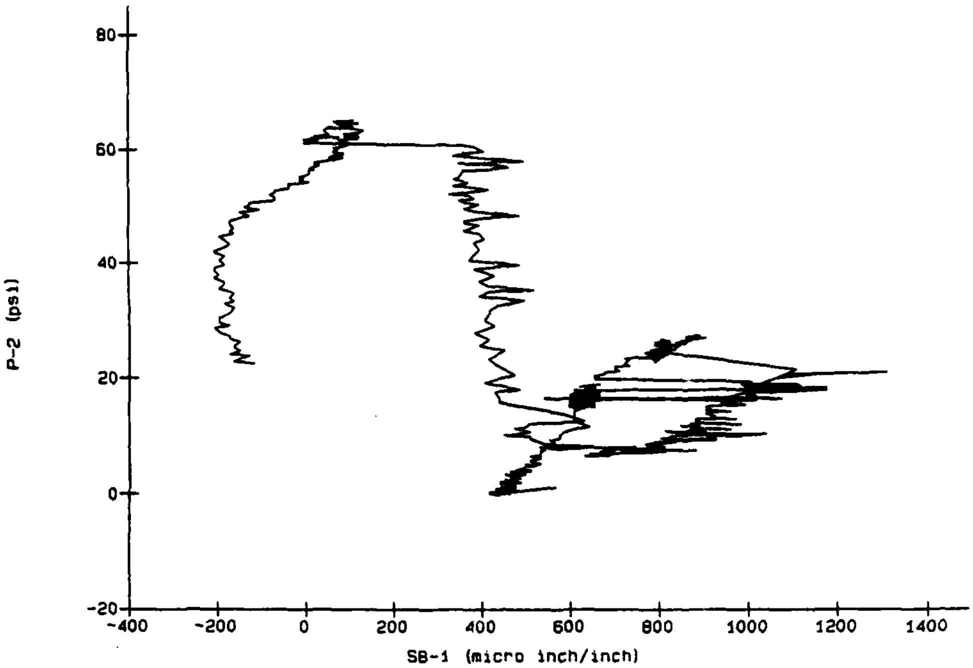

SLAB 7
120
100
80
60
40
20
0
-20
P-2 (psi)
-4000
-2000
0
2000
4000
6000
8000
1.000E+4
1.200E+4
SL-6 (micro inch/inch)


SLAB 8
100
80
60
40
20
0
P-2 (psi)
0
50
100
150
200
250
300
350
400
450
500
TIME (seconds)


SLAB 8
P-2 (psi)
80
60
40
20
0
-20
0.0
0.50
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.00
5.50
D-1 (inches)

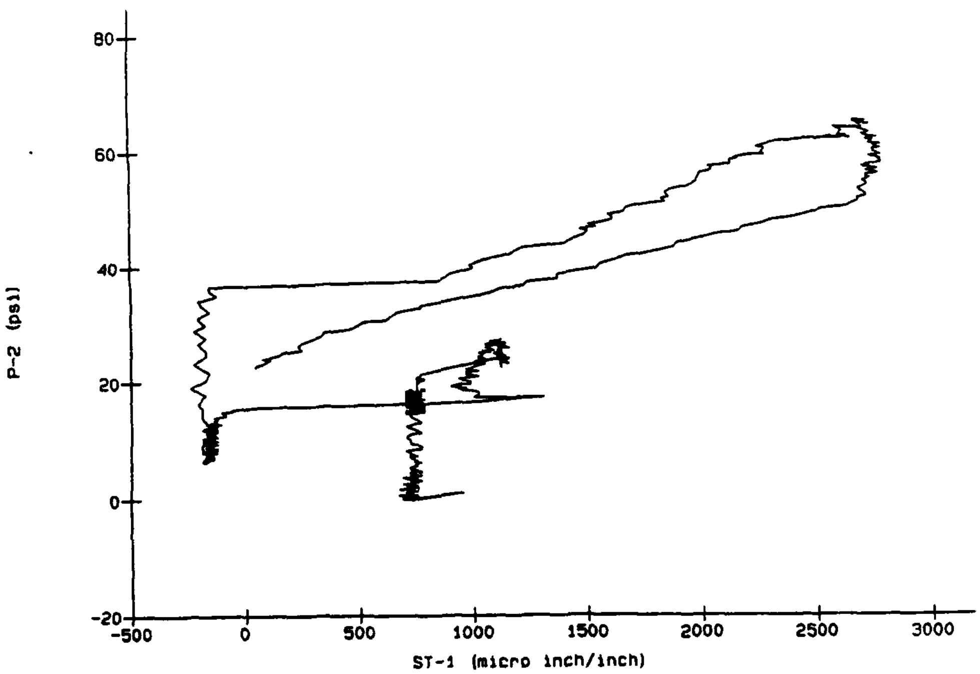

SLAB 8
P-2 (psi)
80
60
40
20
0
-20
0.0
0.50
1.0
1.5
2.0
2.5
D-2 (inches)

SLAB 8



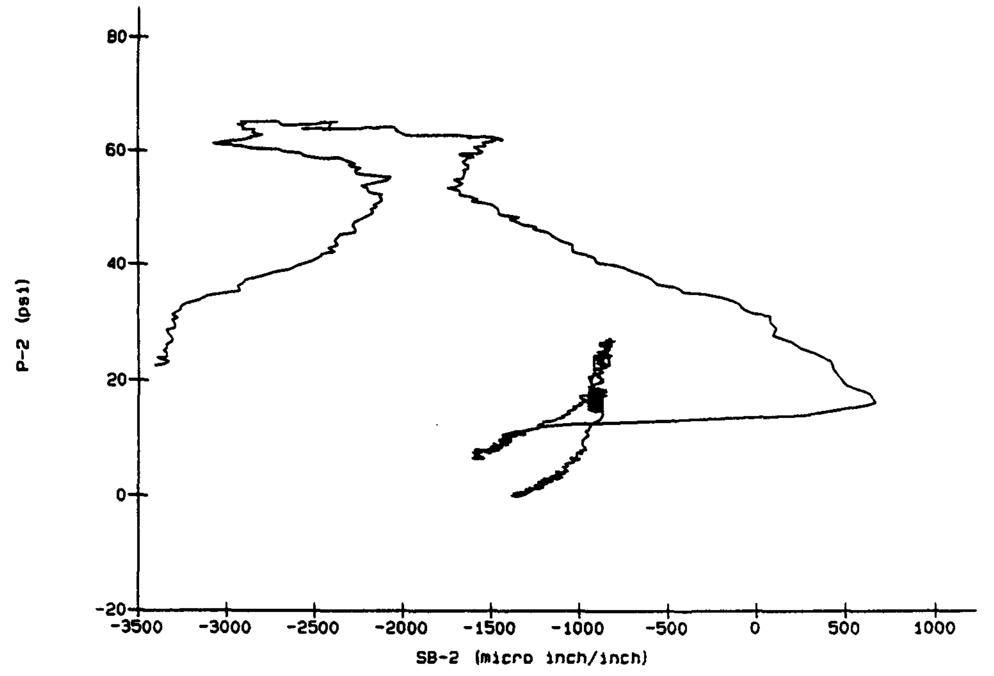
80
60
40
20
0
-20
P-2 (psi)
-400
-200
0
200
400
600
800
1000
1200
1400
SB-1 (micro inch/inch)

SLAB 8



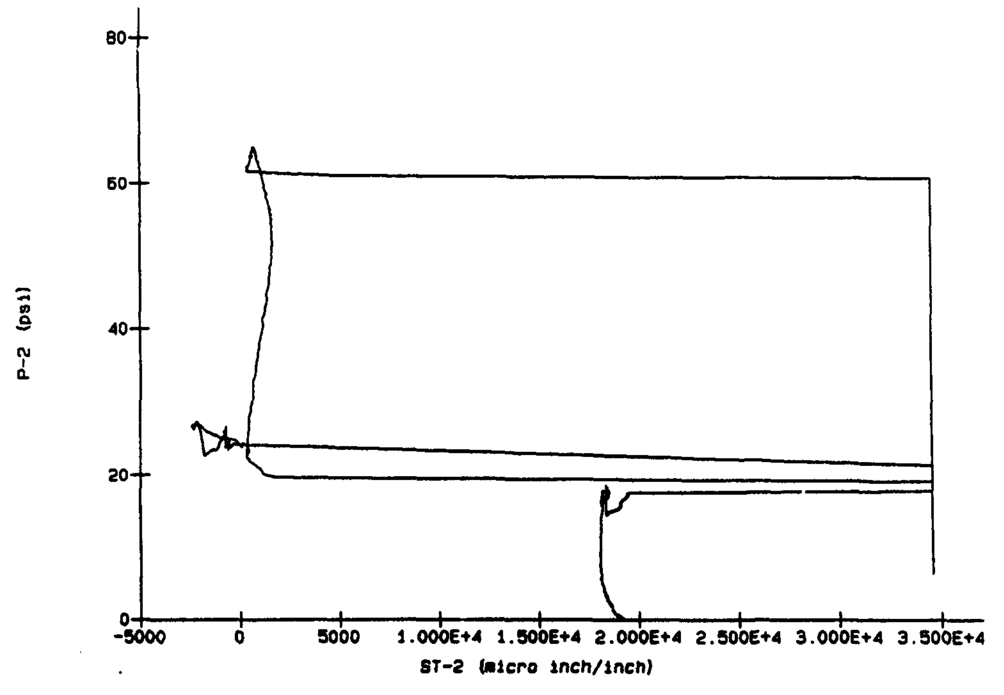
80
60
40
20
0
-20
P-2 (psi)
-500
0
500
1000
1500
2000
2500
3000
ST-1 (micro inch/inch)

SLAB 8

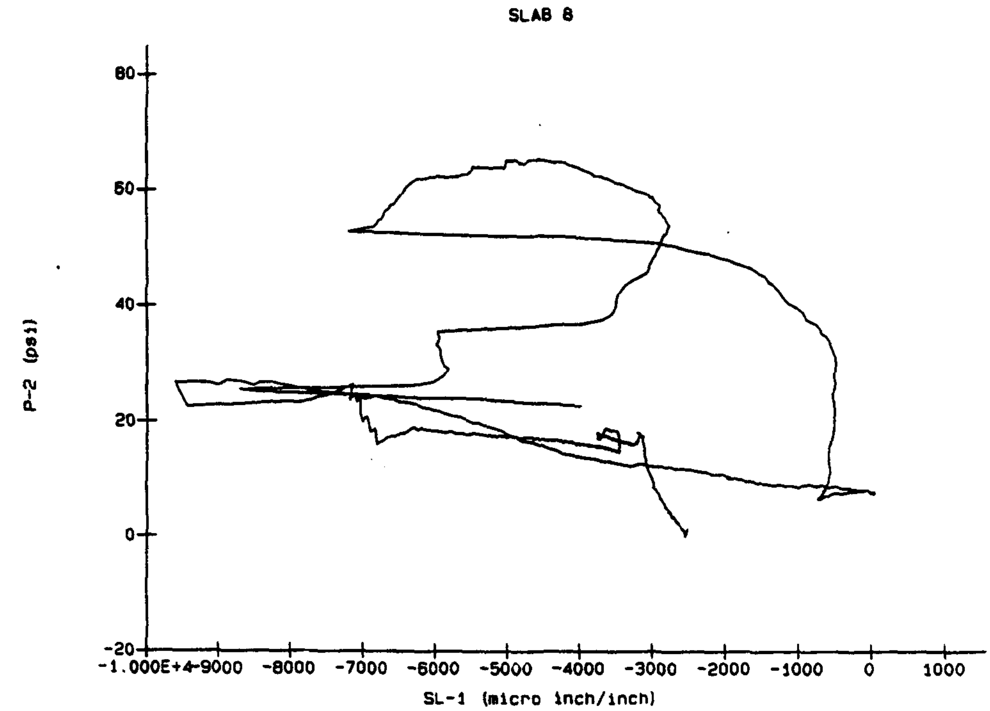


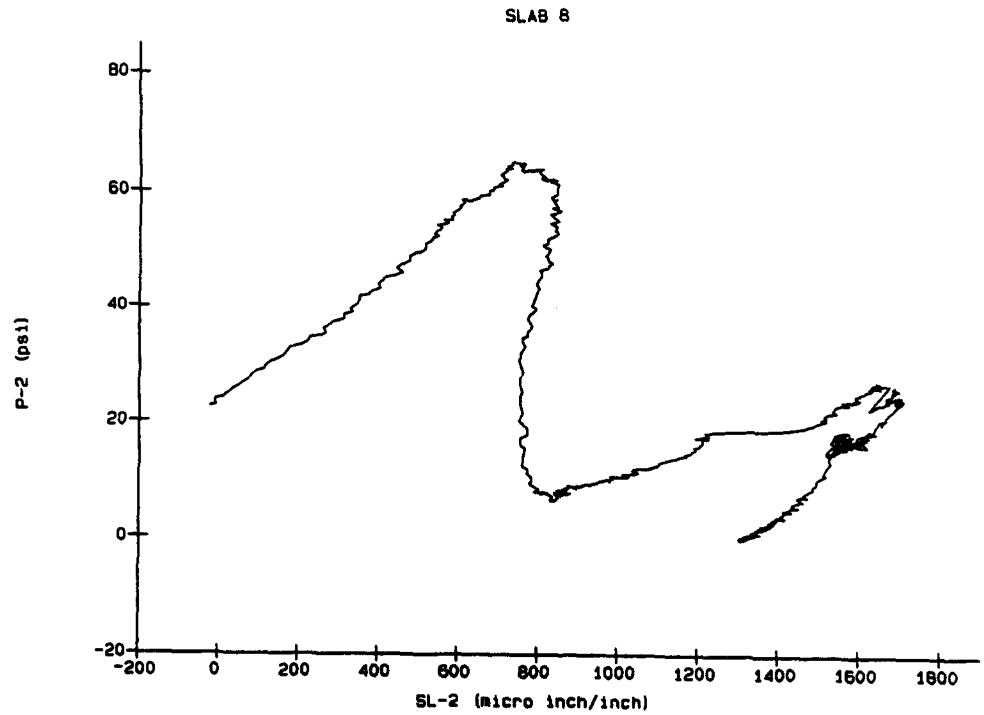
P-2 (psi)
80
60
40
20
0
-20
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
SB-2 (micro inch/inch)

SLAB 8

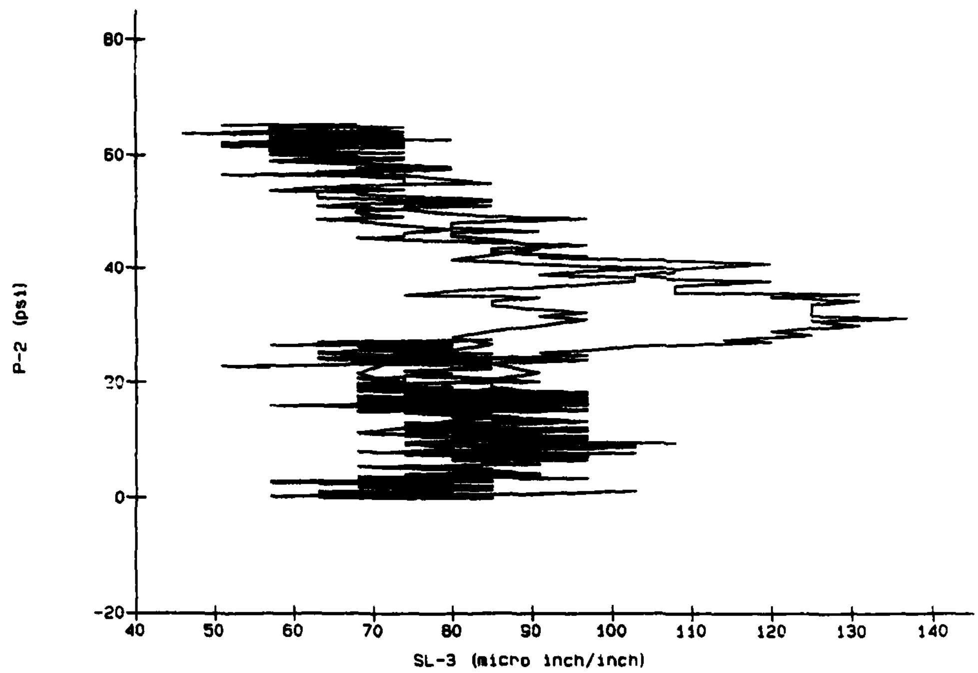


P-2 (psi)
80
60
40
20
0
-5000
0
5000
1.000E+4
1.500E+4
2.000E+4
2.500E+4
3.000E+4
3.500E+4
ST-2 (micro inch/inch)


SLAB 8
P-2 (psi)
80
60
40
20
0
-20
-1.000E+4
-9000
-8000
-7000
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
SL-1 (micro inch/inch)

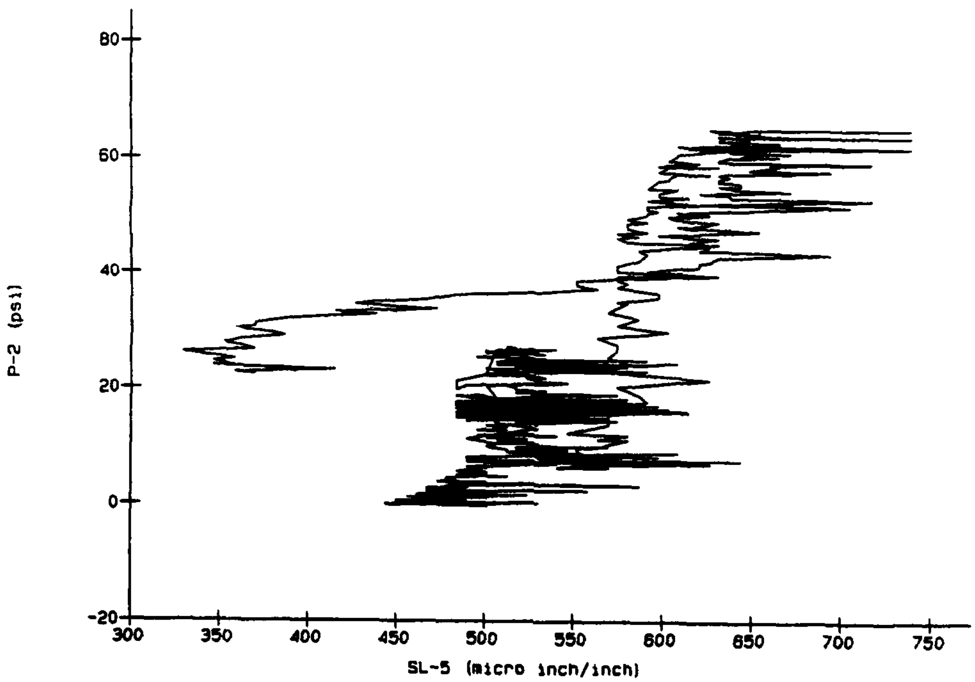

SLAB 8
P-2 (psi)
80
60
40
20
0
-20
-200
0
200
400
600
800
1000
1200
1400
1600
1800
SL-2 (micro inch/inch)

SLAB 8



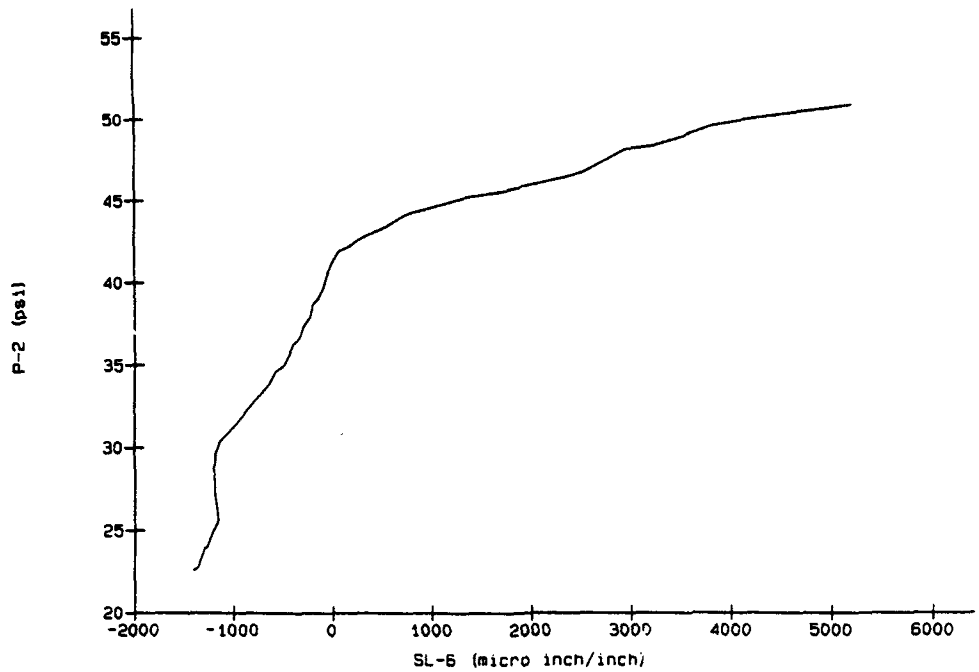
80
60
40
20
0
-20
P-2 (psi)
40
50
60
70
80
90
100
110
120
130
140
SL-3 (micro inch/inch)

SLAB 8



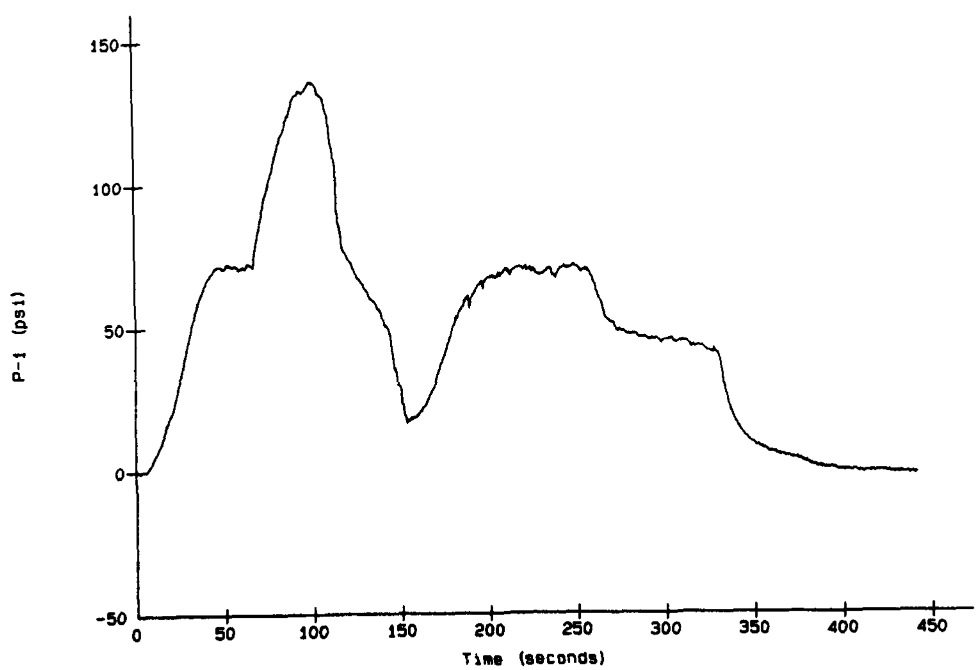
80
60
40
20
0
-20
P-2 (psi)
300
350
400
450
500
550
600
650
700
750
SL-5 (micro inch/inch)

SLAB 8



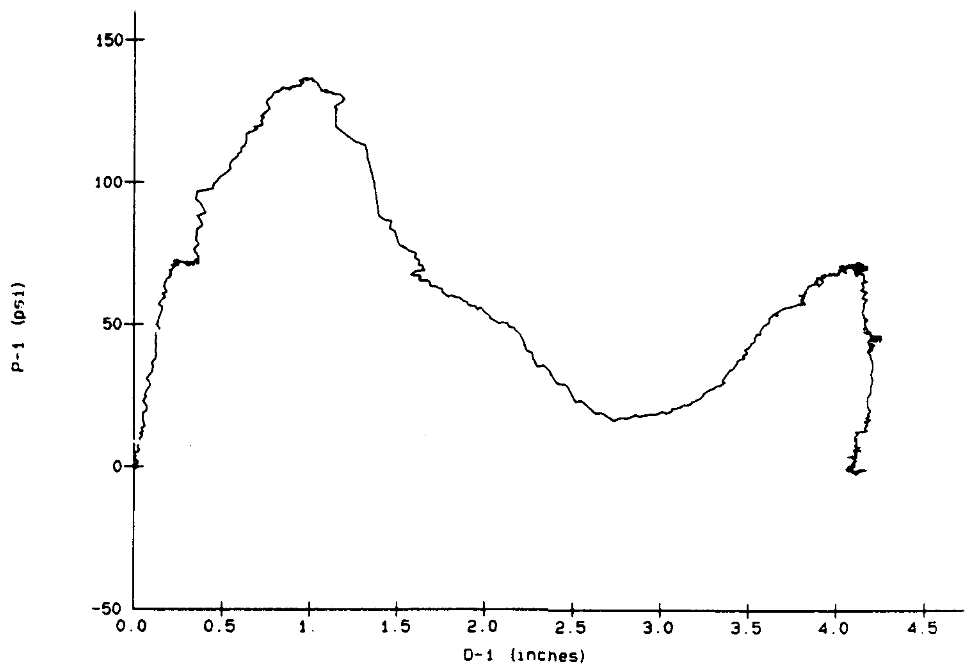
55
50
45
40
35
30
25
20
P-2 (psi)
-2000
-1000
0
1000
2000
3000
4000
5000
6000
SL-6 (micro inch/inch)

Slab 9



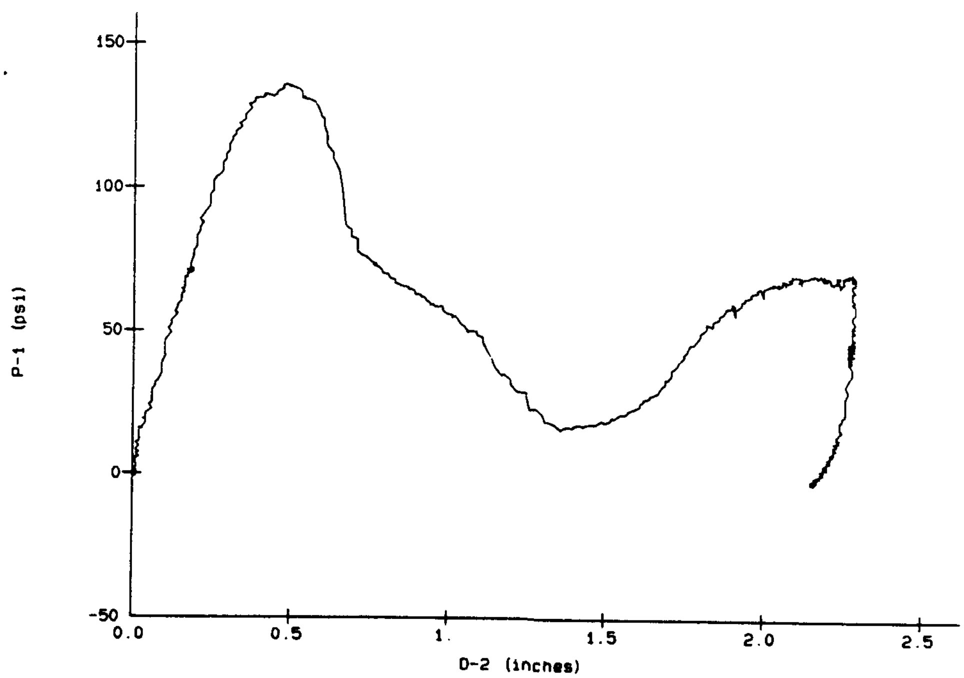
150
100
50
0
-50
P-1 (psi)
0
50
100
150
200
250
300
350
400
450
Time (seconds)

Slab 9



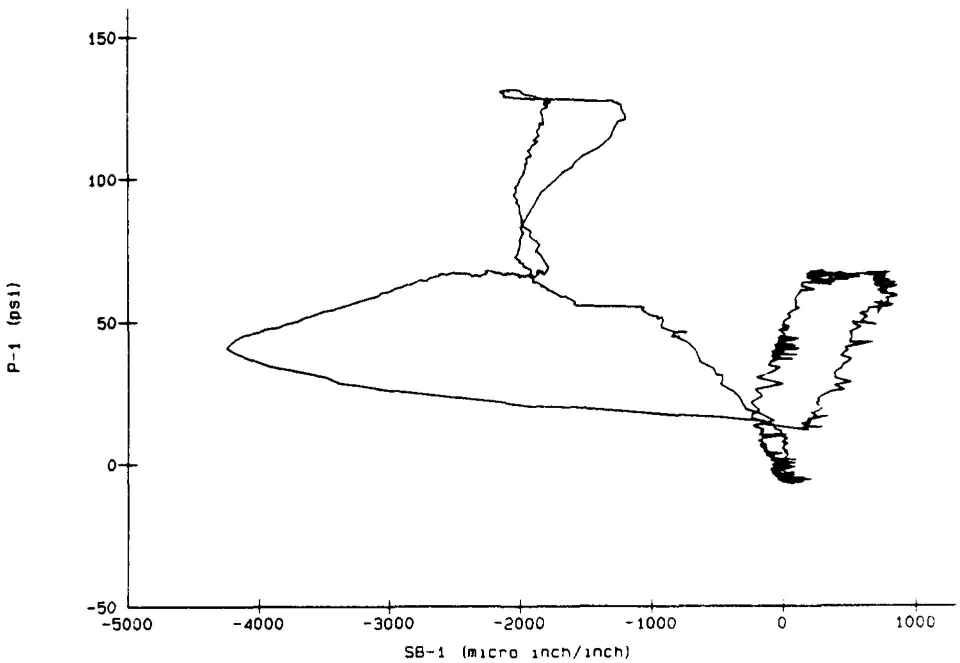
150
100
50
0
-50
P-1 (psi)
0.0
0.5
1.
1.5
2.0
2.5
3.0
3.5
4.0
4.5
D-1 (inches)

Slab 9

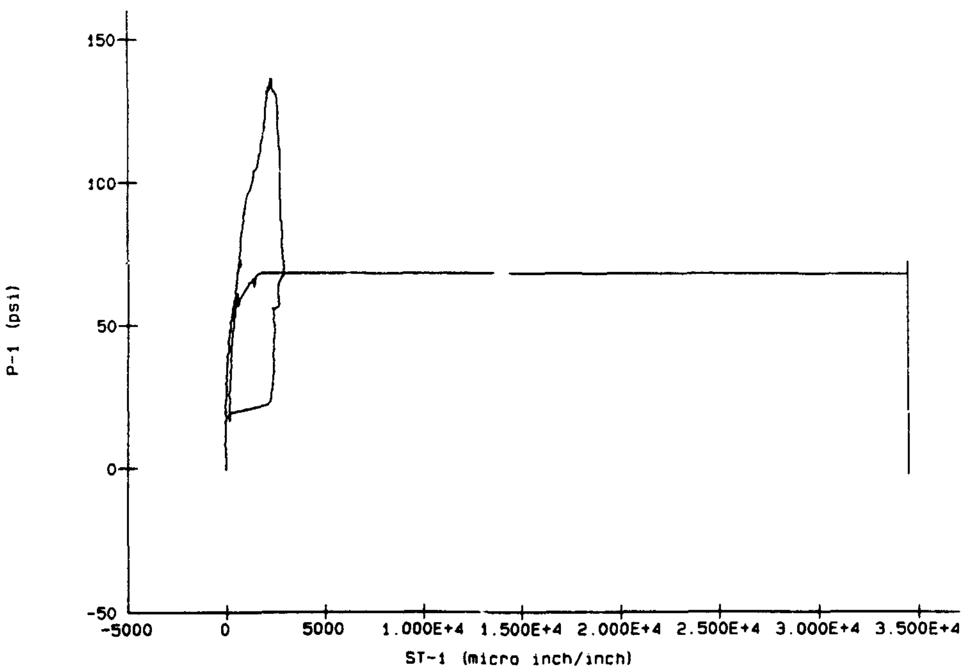


150
100
50
0
-50
P-1 (psi)
0.0
0.5
1.
1.5
2.0
2.5
D-2 (inches)

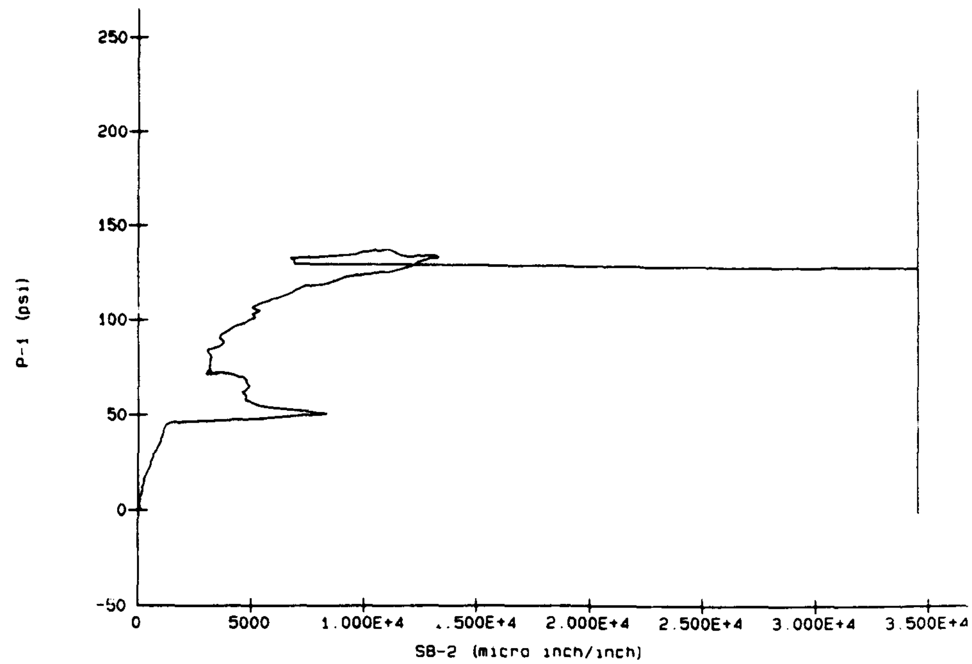
Slab 9


150
100
50
0
-50
P-1 (psi)
-5000
-4000
-3000
-2000
-1000
0
1000
SB-1 (micro inch/inch)

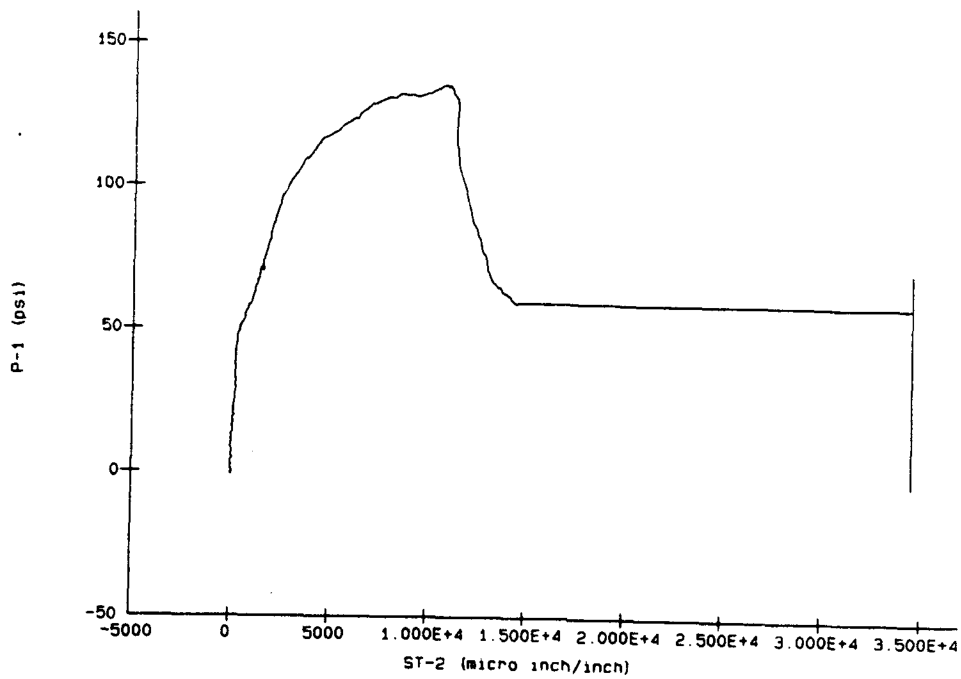
Slab 9

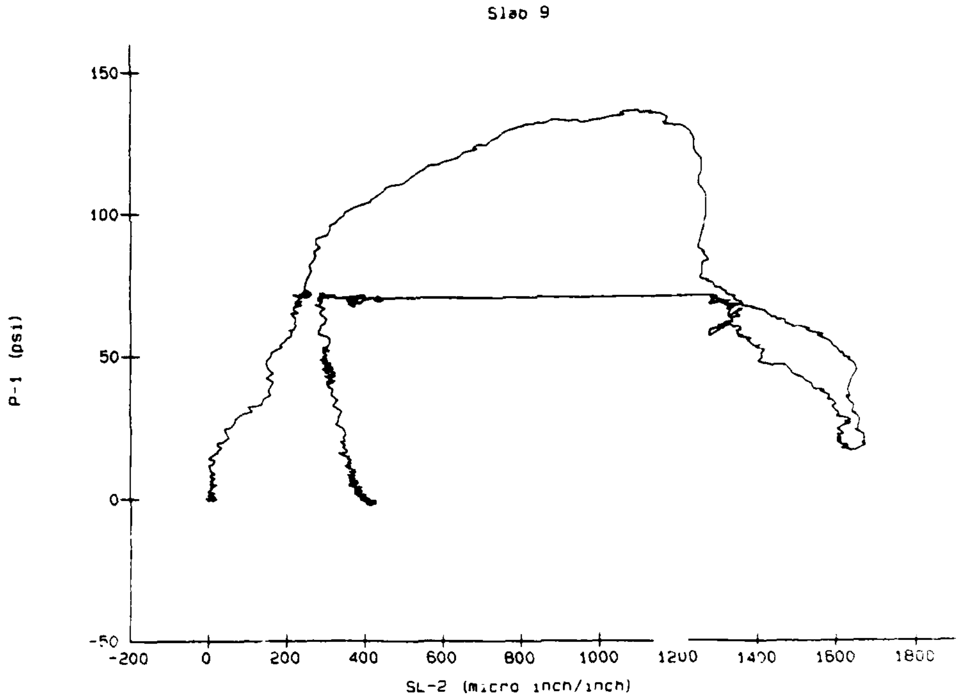

150
100
50
0
-50
P-1 (psi)
-5000
0
5000
1.000E+4
1.500E+4
2.000E+4
2.500E+4
3.000E+4
3.500E+4
ST-1 (micro inch/inch)

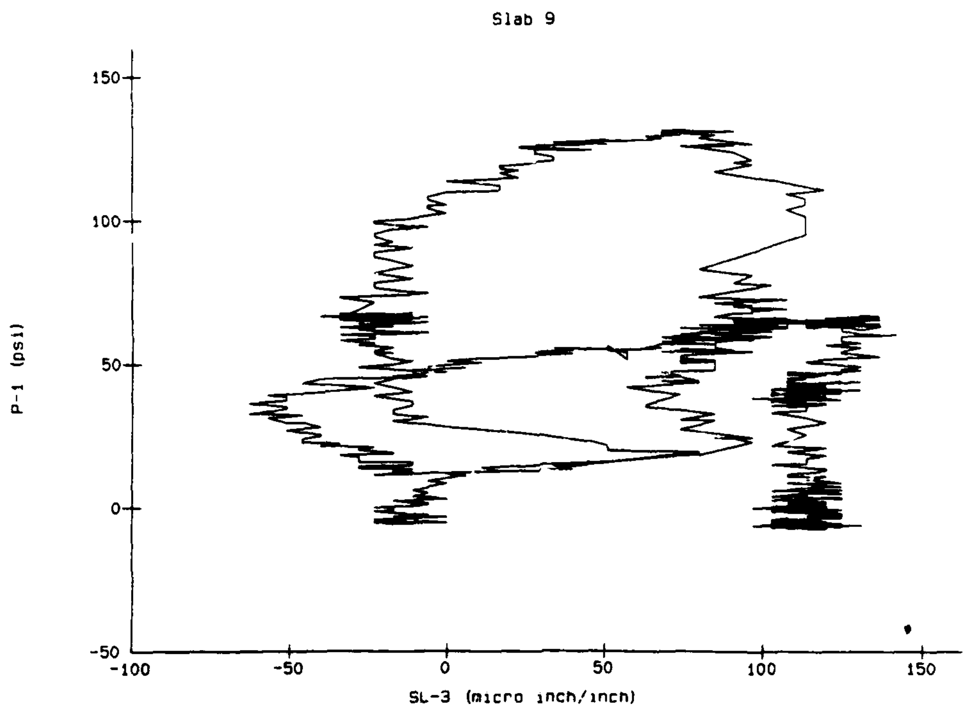
Slab 9


P-1 (psi)
250
200
150
100
50
0
-50
0
5000
1.000E+4
.500E+4
2.000E+4
2.500E+4
3.000E+4
3.500E+4
SB-2 (micro inch/inch)

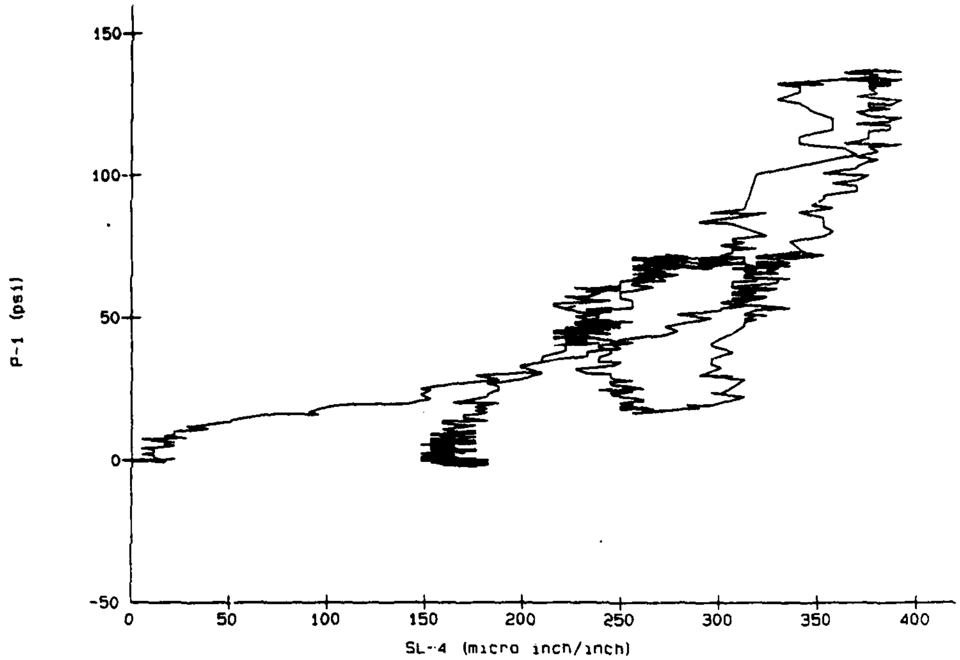
Slab 9


P-1 (psi)
150
100
50
0
-50
-5000
0
5000
1.000E+4
1.500E+4
2.000E+4
2.500E+4
3.000E+4
3.500E+4
ST-2 (micro inch/inch)

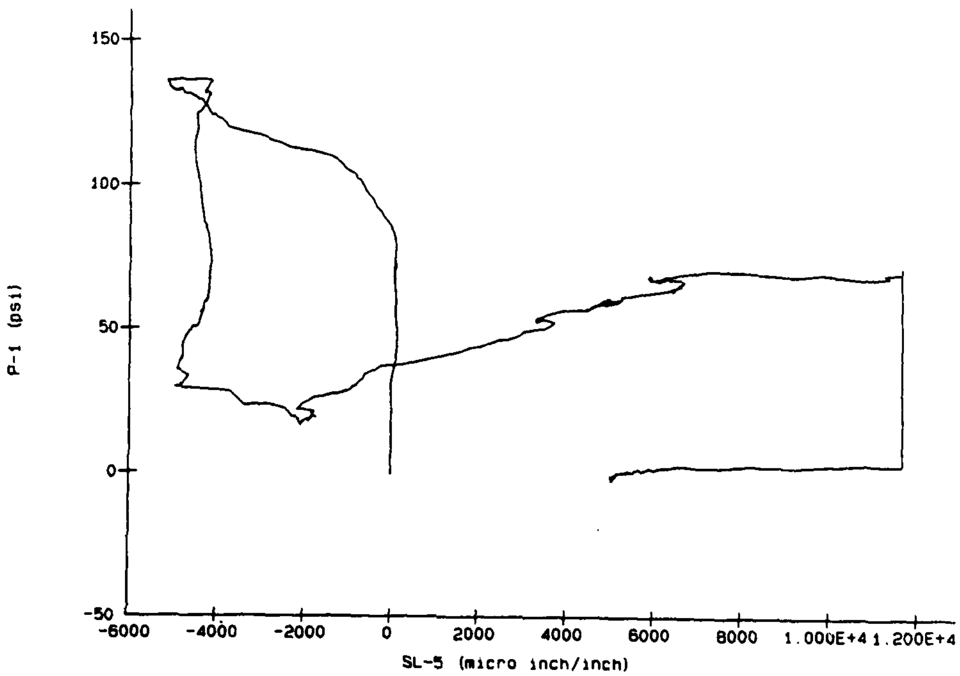

Slab 9
P-1 (psi)
150
100
50
0
-50
-200
0
200
400
600
800
1000
1200
1400
1600
1800
SL-2 (micro inch/inch)


Slab 9
P-1 (psi)
150
100
50
0
-50
-100
-50
0
50
100
150
SL-3 (micro inch/inch)

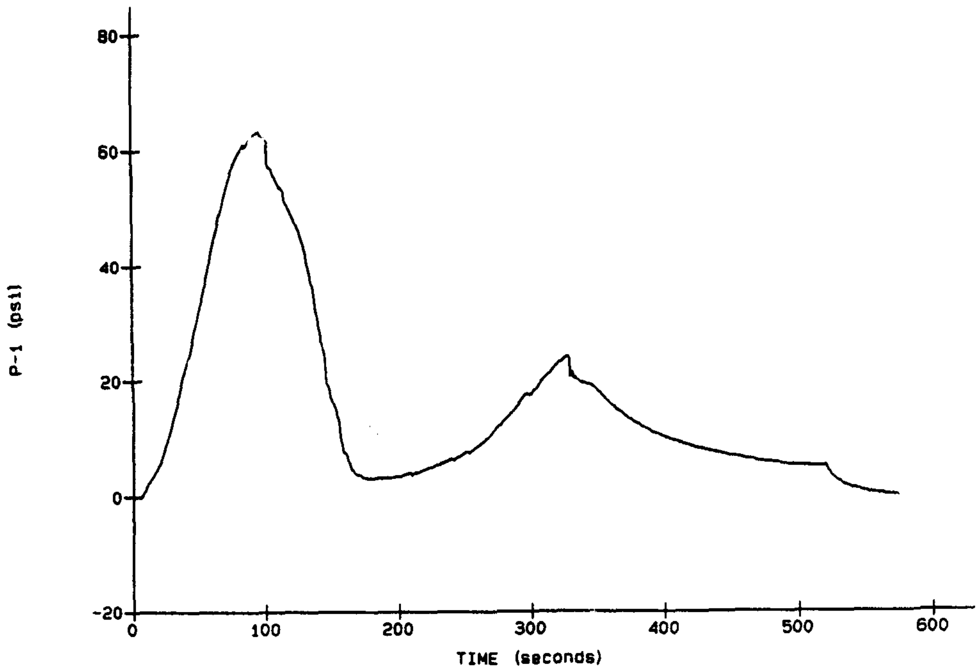
Slab 9


P-1 (psi)
150
100
50
0
-50
0
50
100
150
200
250
300
350
400
SL-4 (micro inch/inch)

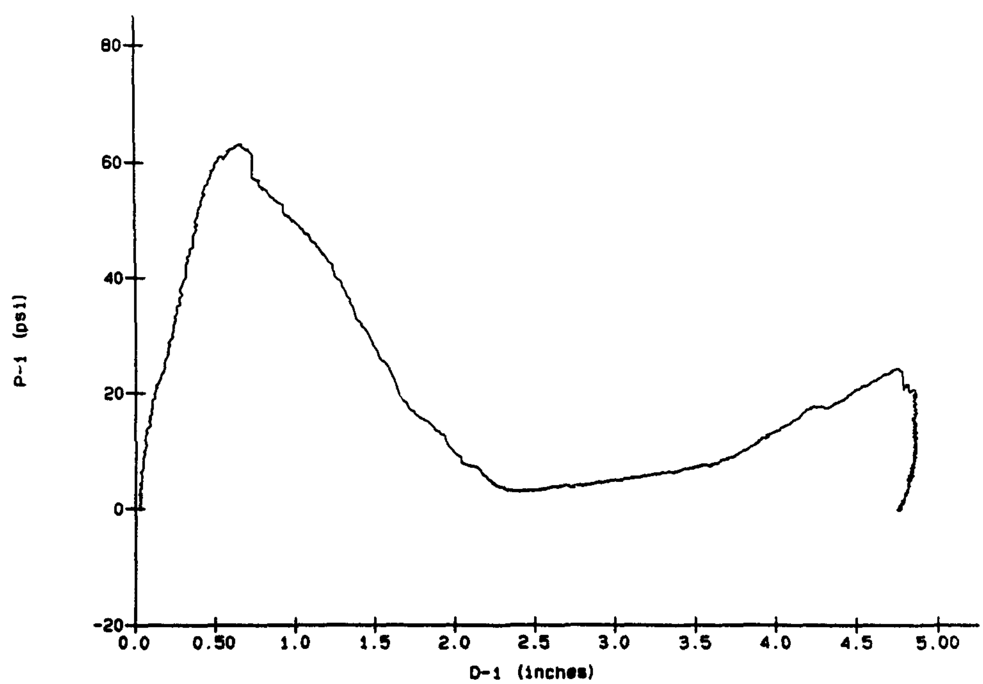
Slab 9

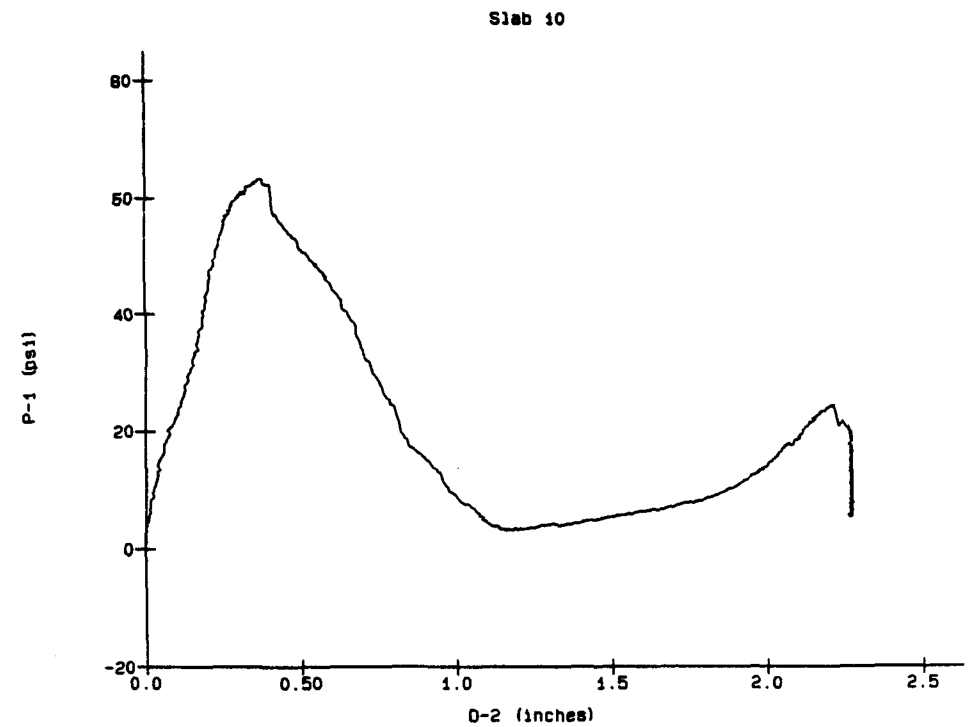

P-1 (psi)
150
100
50
0
-50
-6000
-4000
-2000
0
2000
4000
6000
8000
1.000E+4
1.200E+4
SL-5 (micro inch/inch)

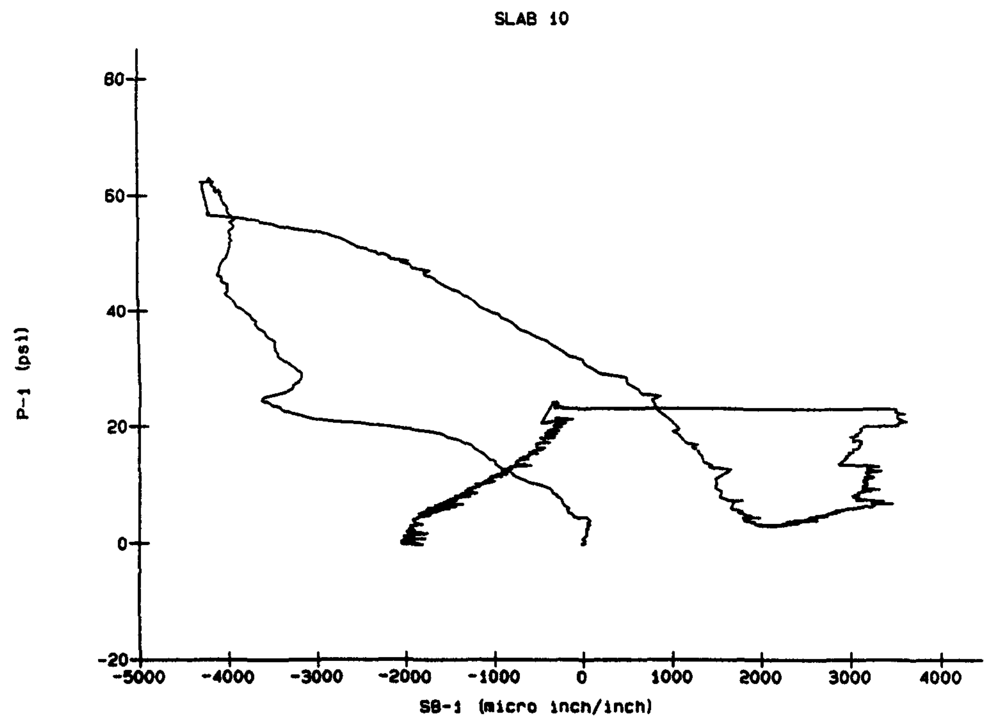
SLAB 10

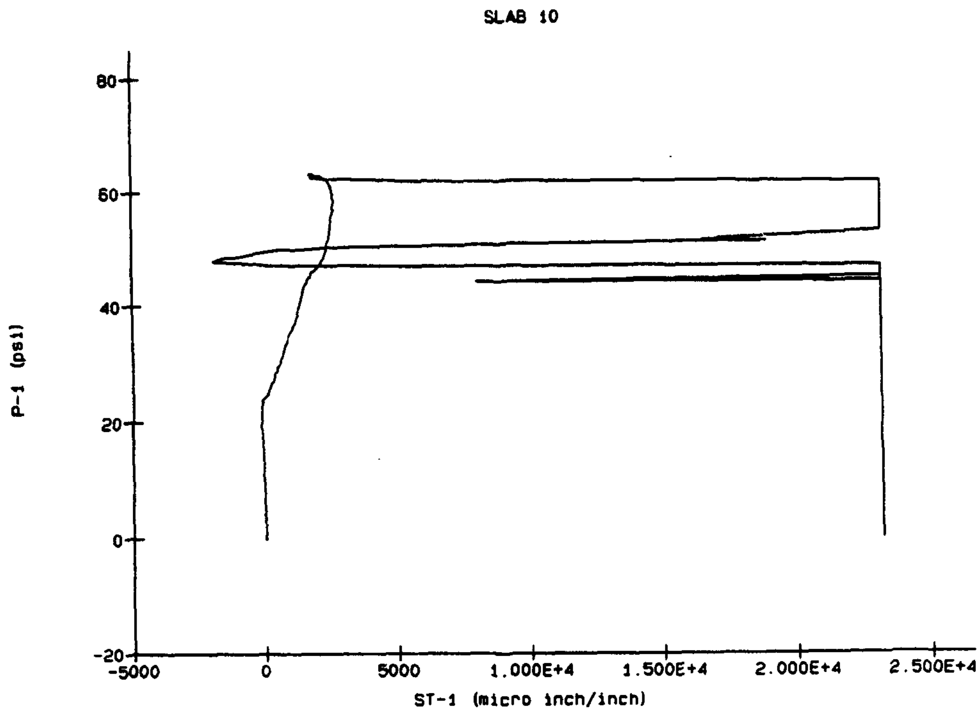


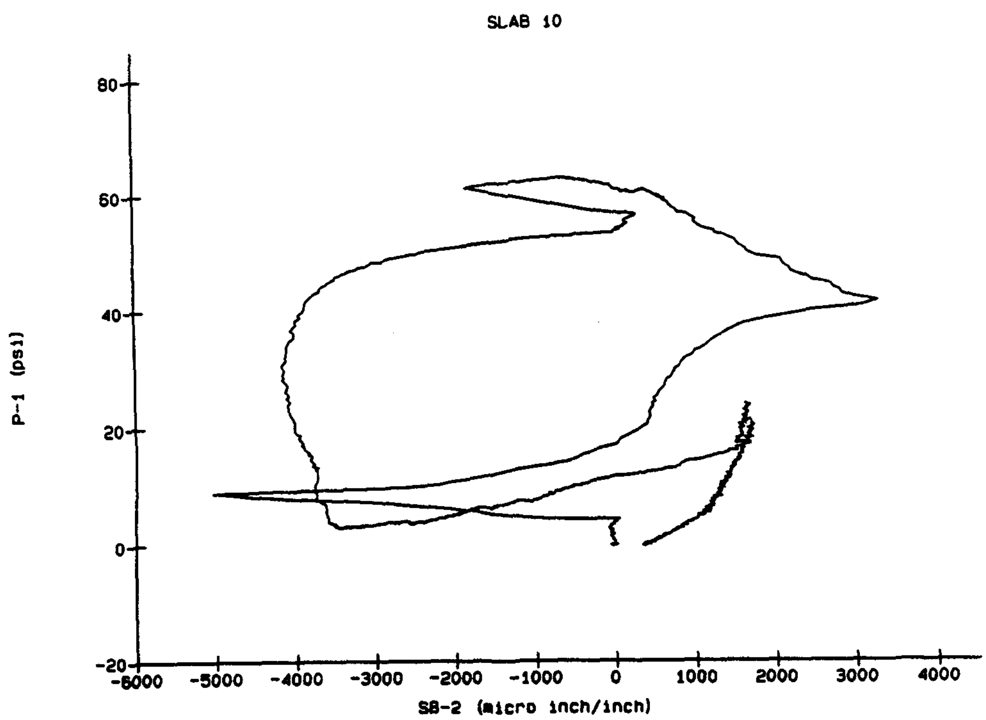
SLAB 10

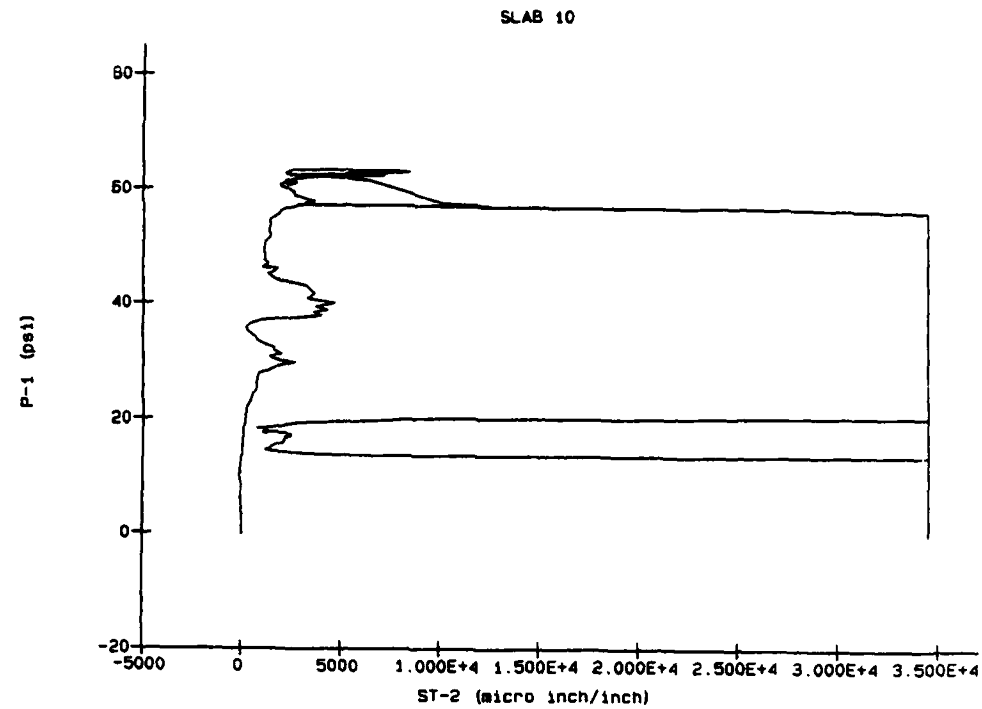


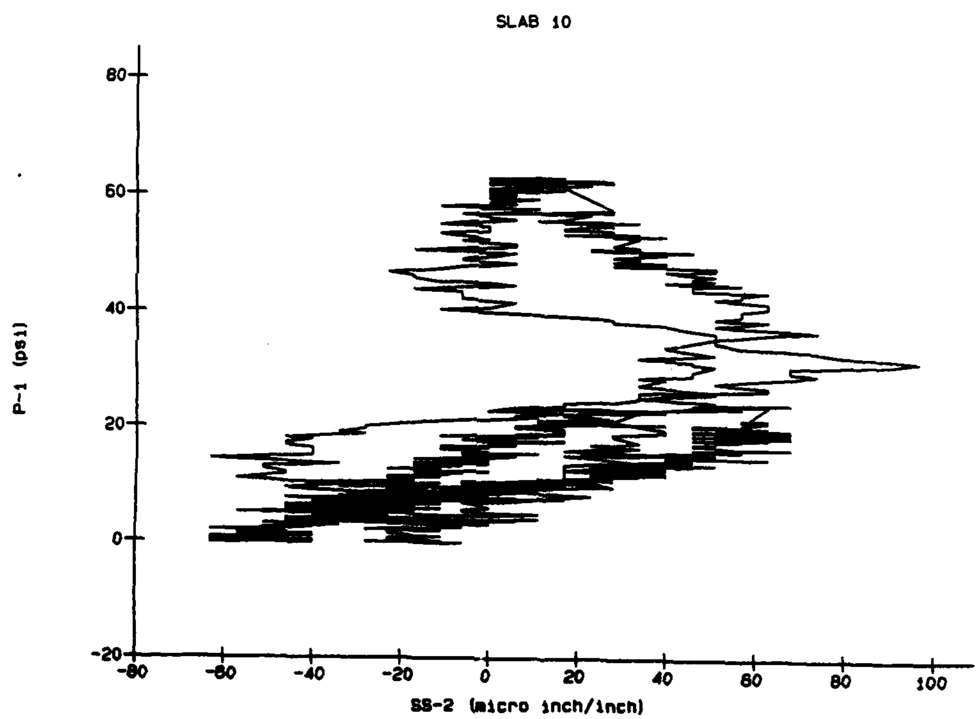

Slab 10
80
60
40
20
0
-20
P-1 (psi)
0.0
0.50
1.0
1.5
2.0
2.5
D-2 (inches)


SLAB 10
80
60
40
20
0
-20
P-1 (psi)
-5000
-4000
-3000
-2000
-1000
0
1000
2000
3000
4000
SB-1 (micro inch/inch)

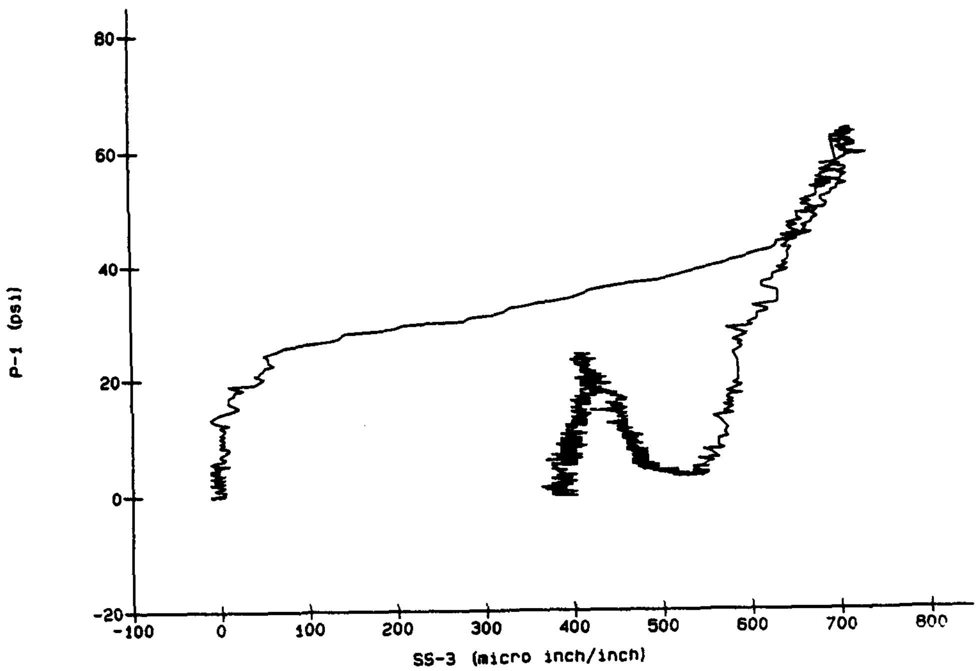

SLAB 10
80
60
40
20
0
-20
P-1 (psi)
-5000
0
5000
1.000E+4
1.500E+4
2.000E+4
2.500E+4
ST-1 (micro inch/inch)


SLAB 10
80
60
40
20
0
-20
P-1 (psi)
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
2000
3000
4000
SB-2 (micro inch/inch)

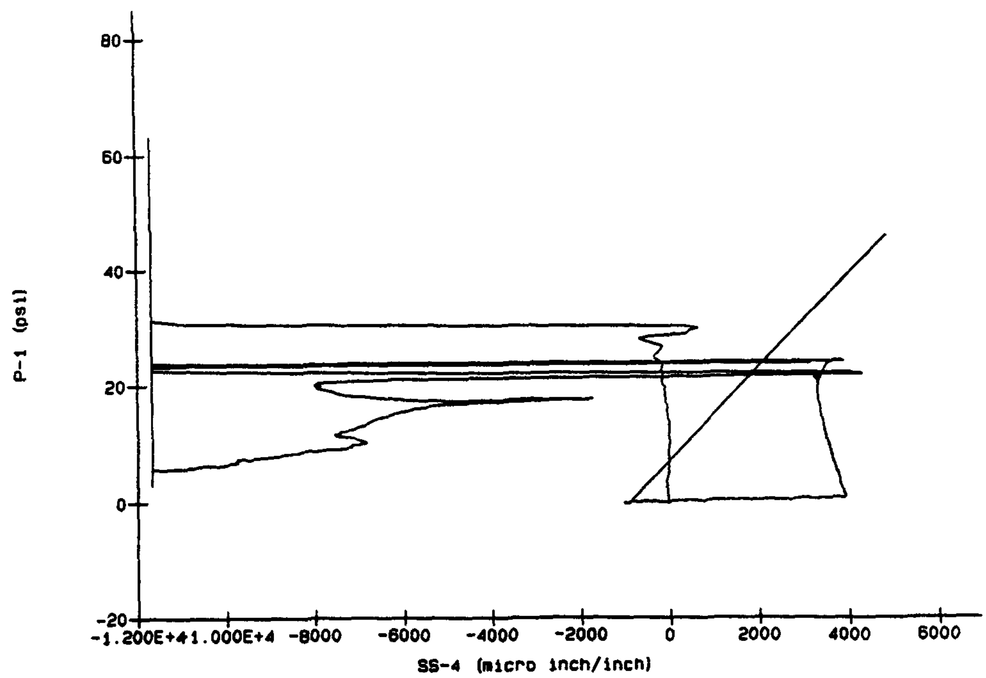

SLAB 10
60
50
40
20
0
-20
P-1 (psi)
-5000
0
5000
1.000E+4
1.500E+4
2.000E+4
2.500E+4
3.000E+4
3.500E+4
ST-2 (micro inch/inch)


SLAB 10
80
60
40
20
0
-20
P-1 (psi)
-80
-60
-40
-20
0
20
40
60
80
100
SS-2 (micro inch/inch)

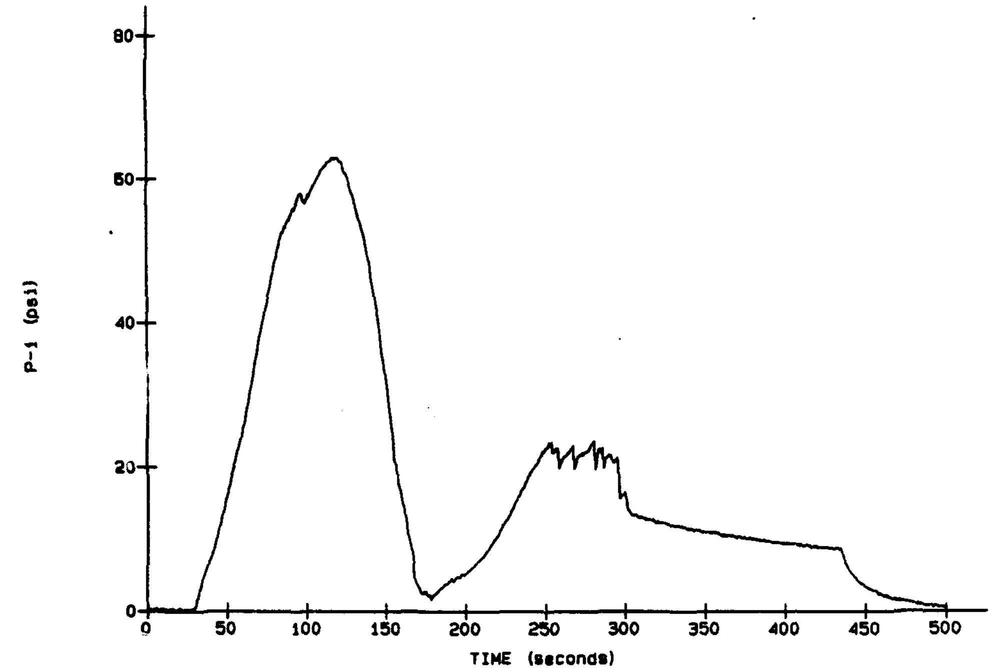
SLAB 10



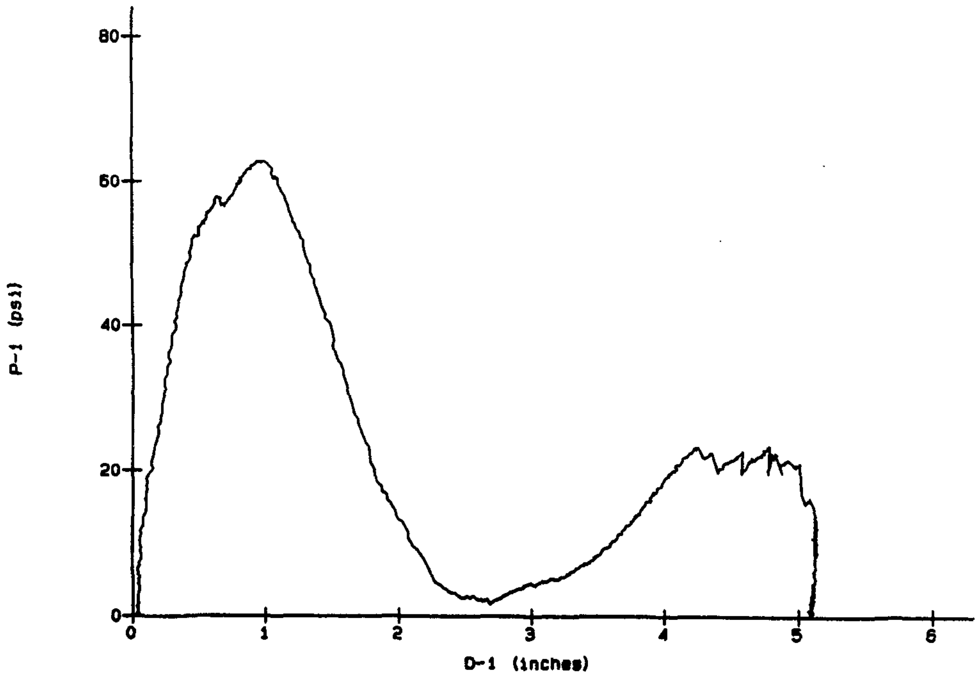
SLAB 10



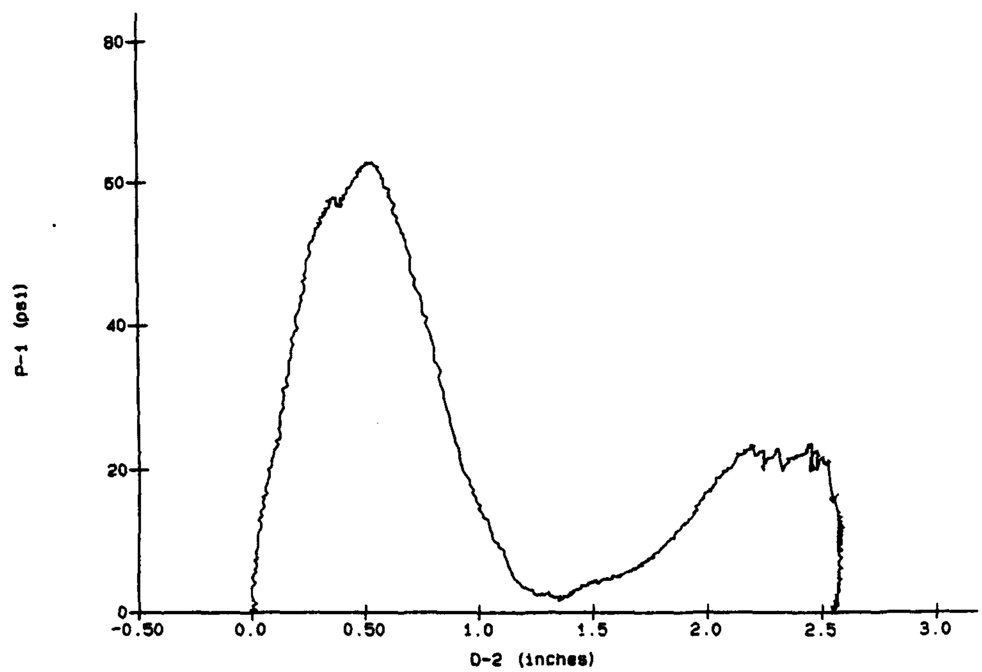
SLAB 11



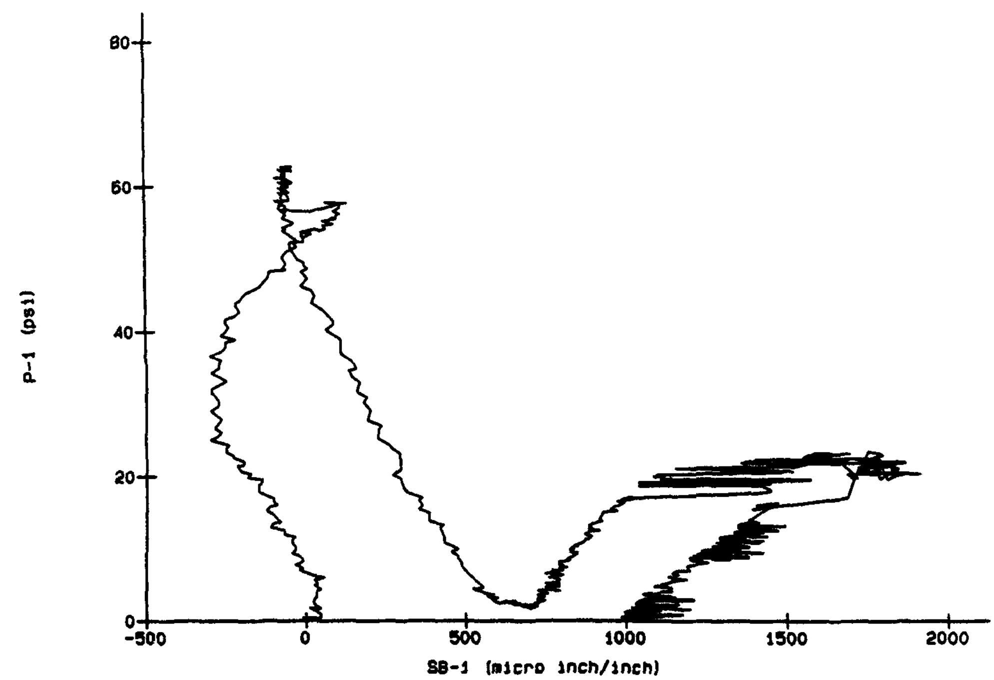
SLAB 11

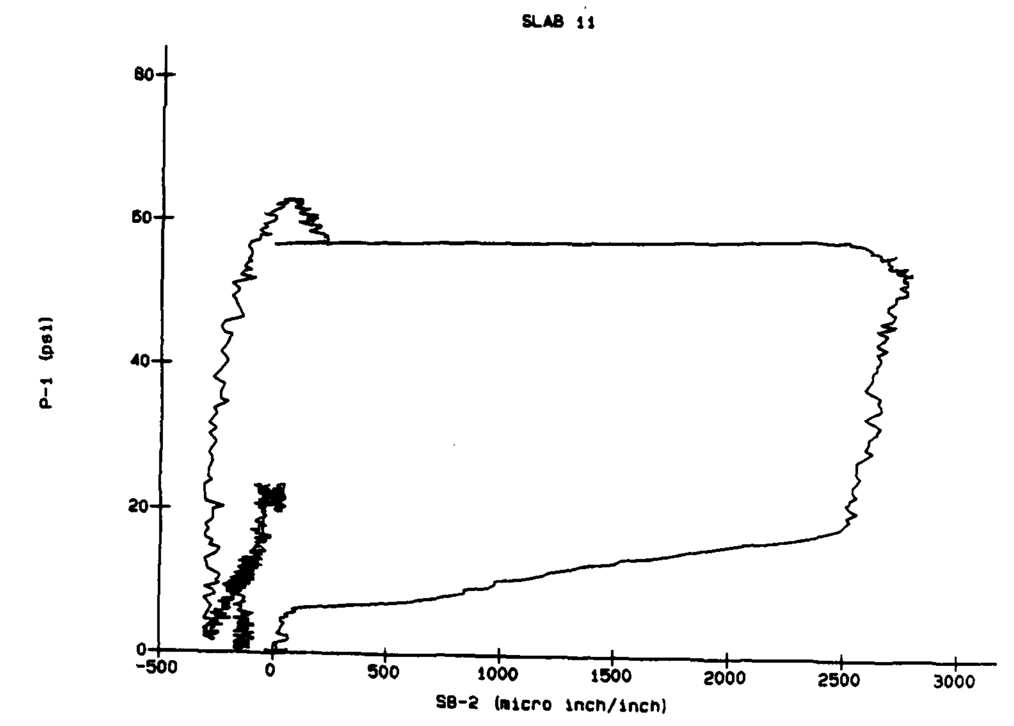


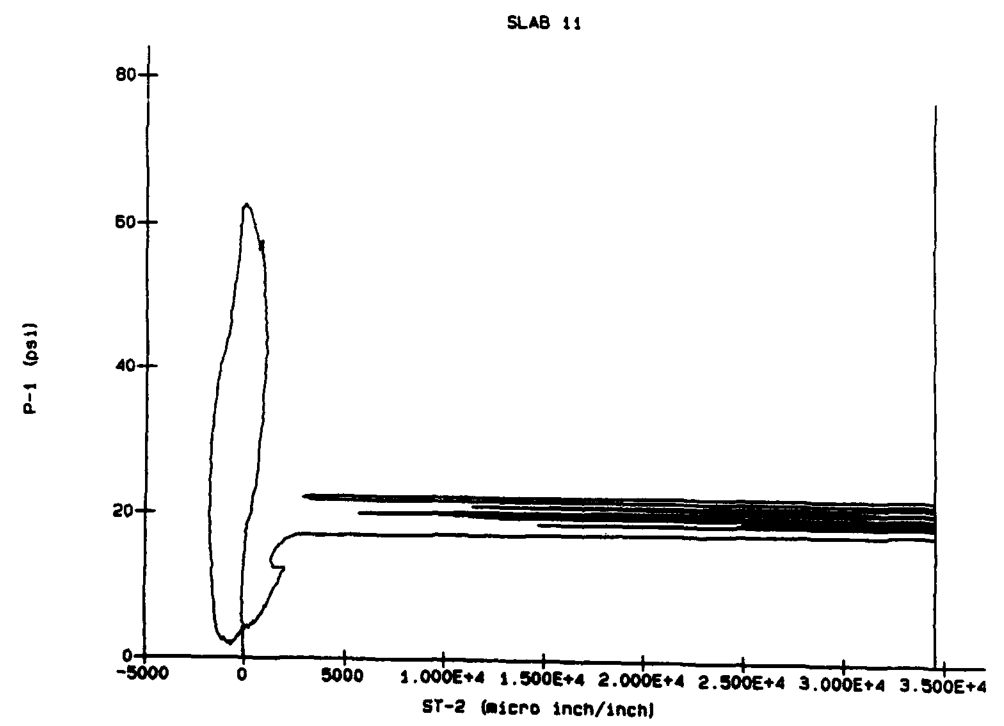
SLAB 11



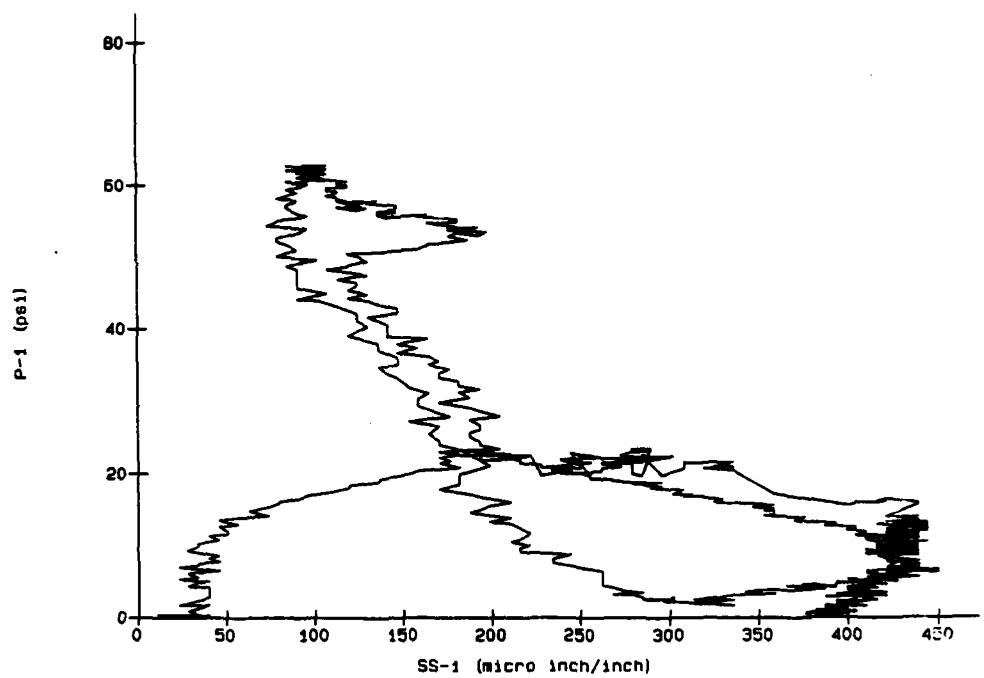
SLAB 11



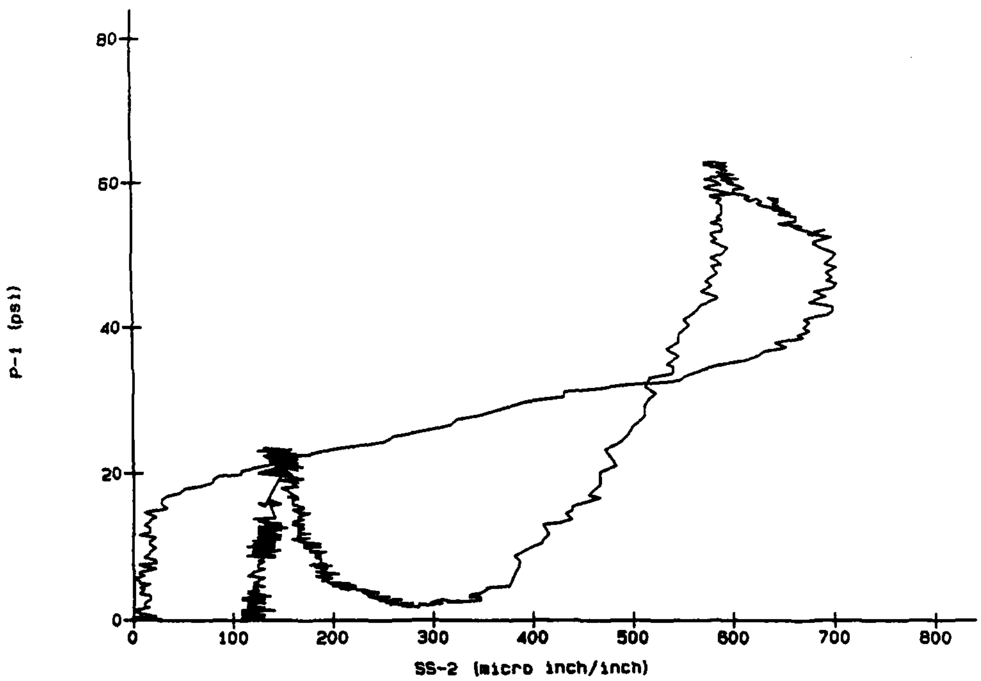

SLAB 11
P-1 (psi)
80
60
40
20
0
-500
0
500
1000
1500
2000
2500
3000
SB-2 (micro inch/inch)

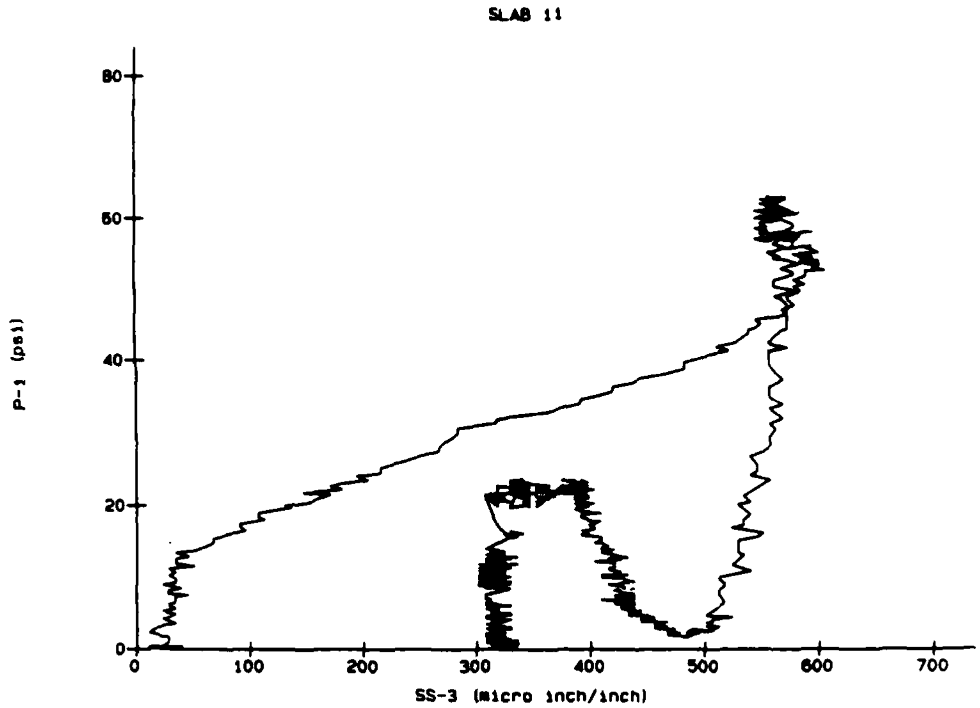

SLAB 11
P-1 (psi)
80
60
40
20
0
-5000
0
5000
1.000E+4
1.500E+4
2.000E+4
2.500E+4
3.000E+4
3.500E+4
ST-2 (micro inch/inch)

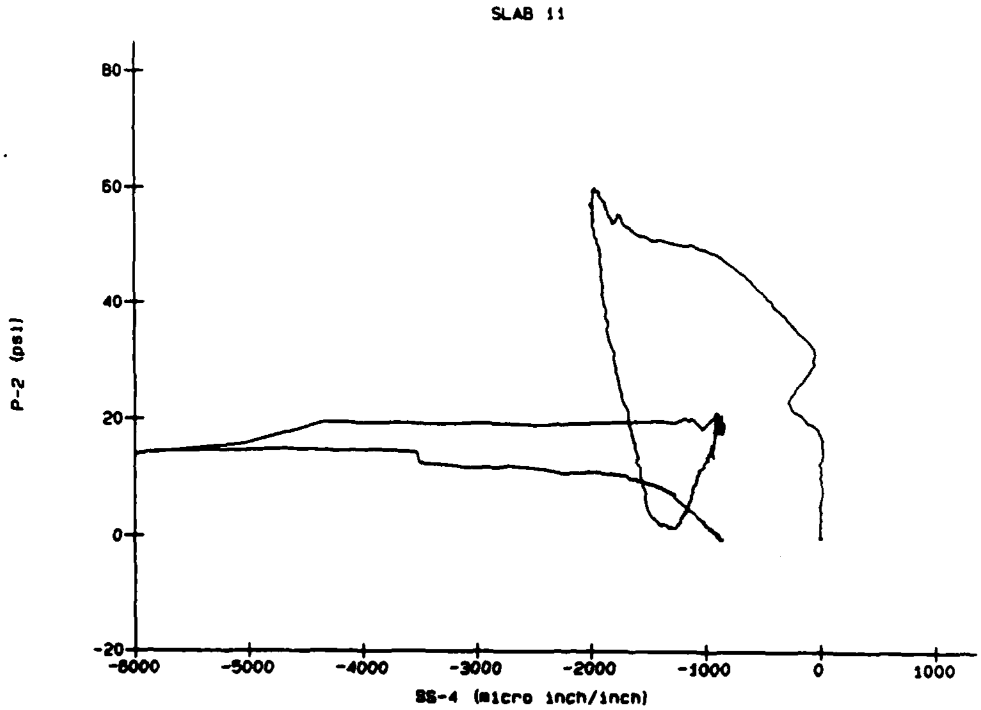
SLAB 11



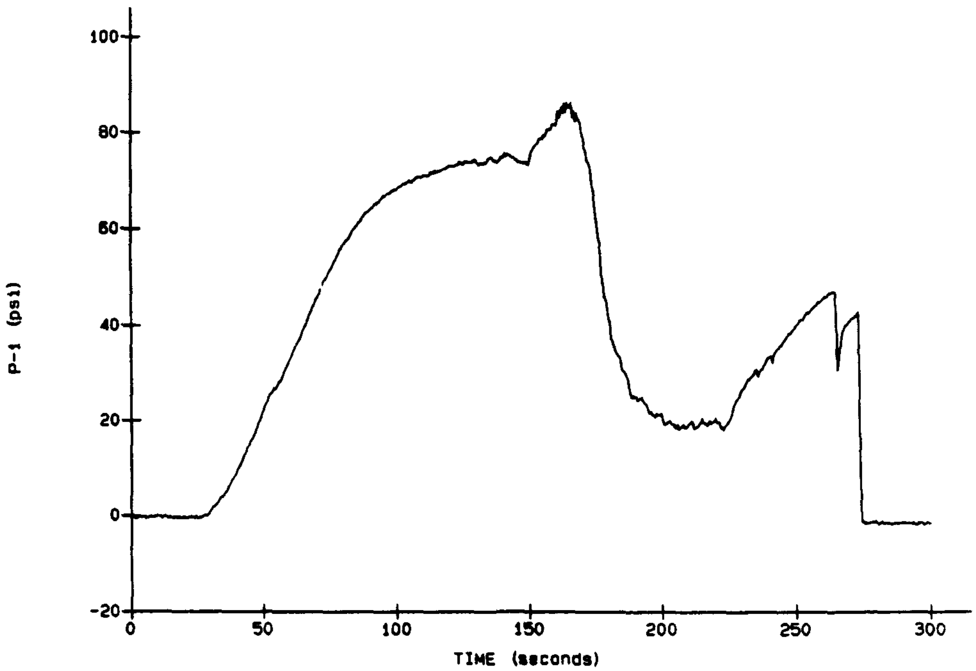
SLAB 11



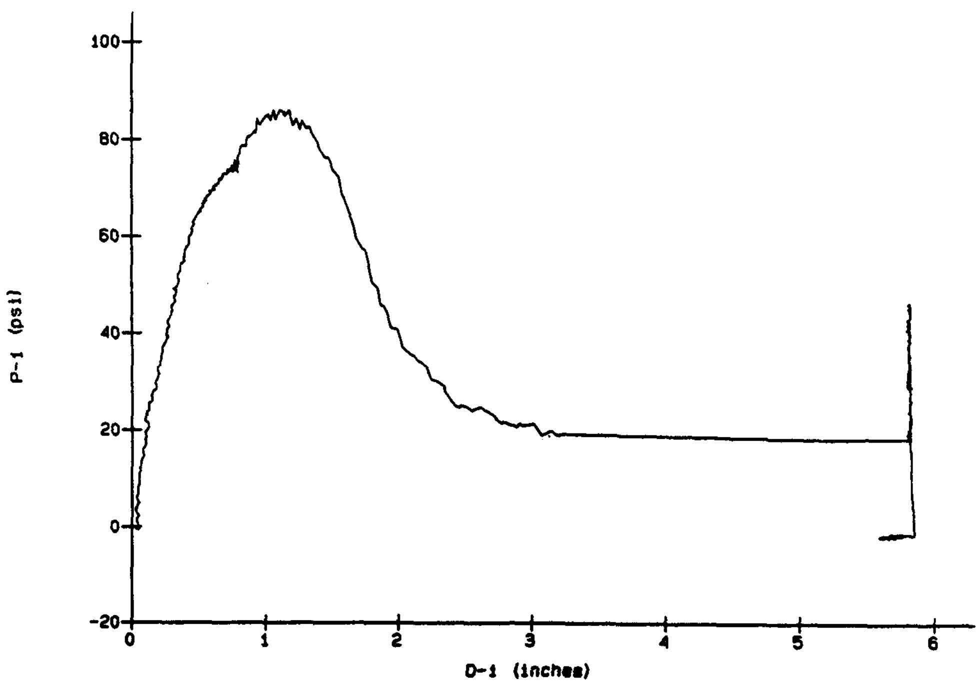

SLAB 11
P-1 (psi)
0
20
40
60
80
SS-3 (micro inch/inch)
0
100
200
300
400
500
600
700

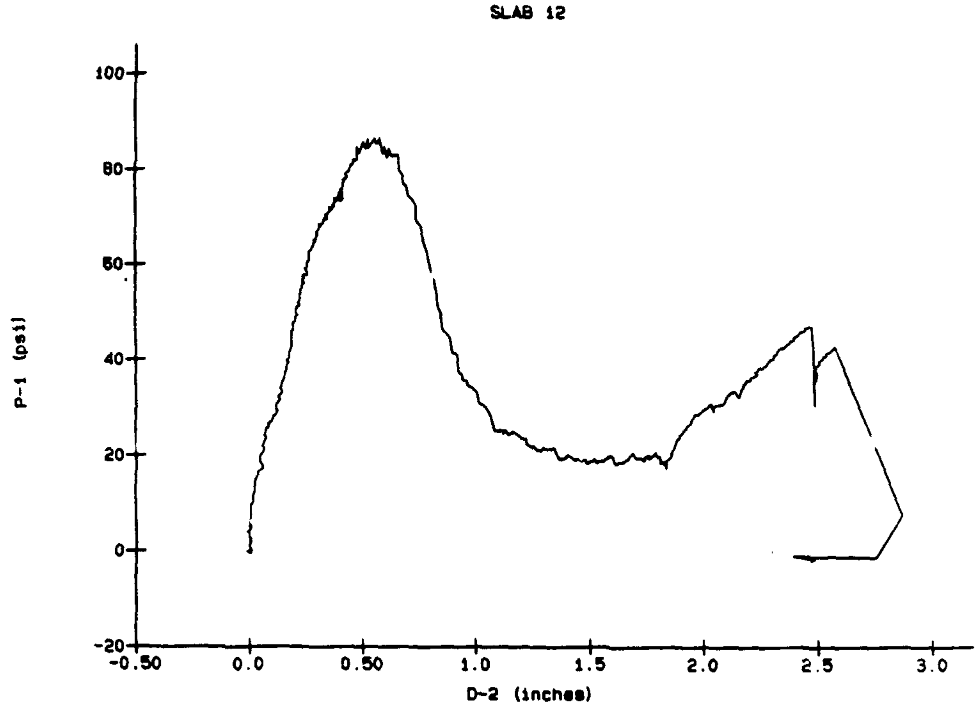

SLAB 11
P-2 (psi)
-20
0
20
40
60
80
SS-4 (micro inch/inch)
-6000
-5000
-4000
-3000
-2000
-1000
0
1000

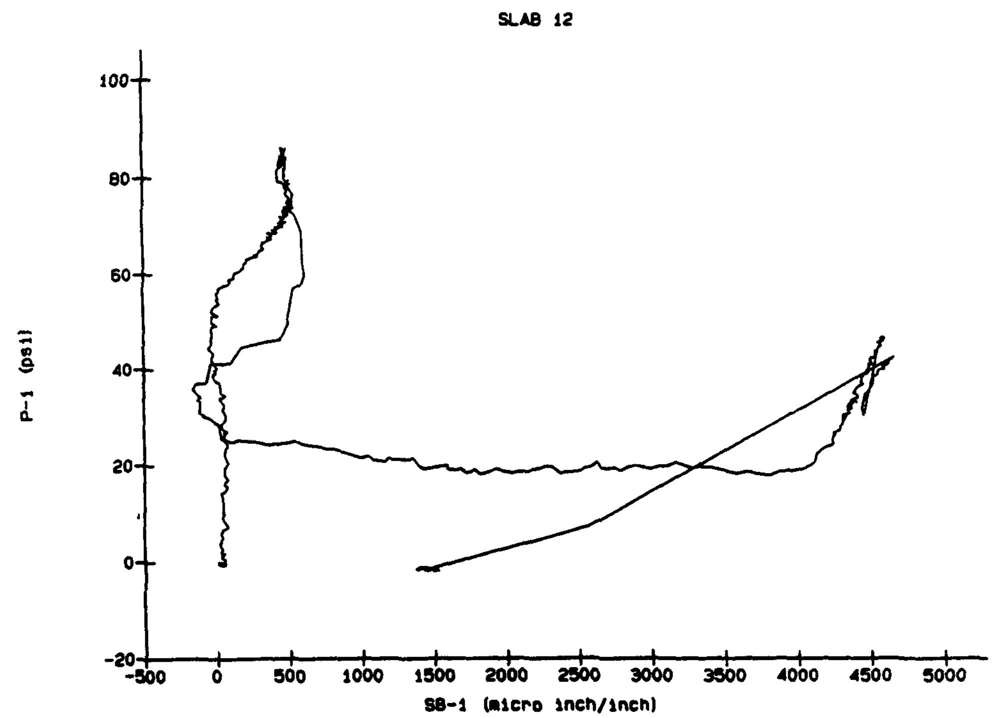
SLAB 12

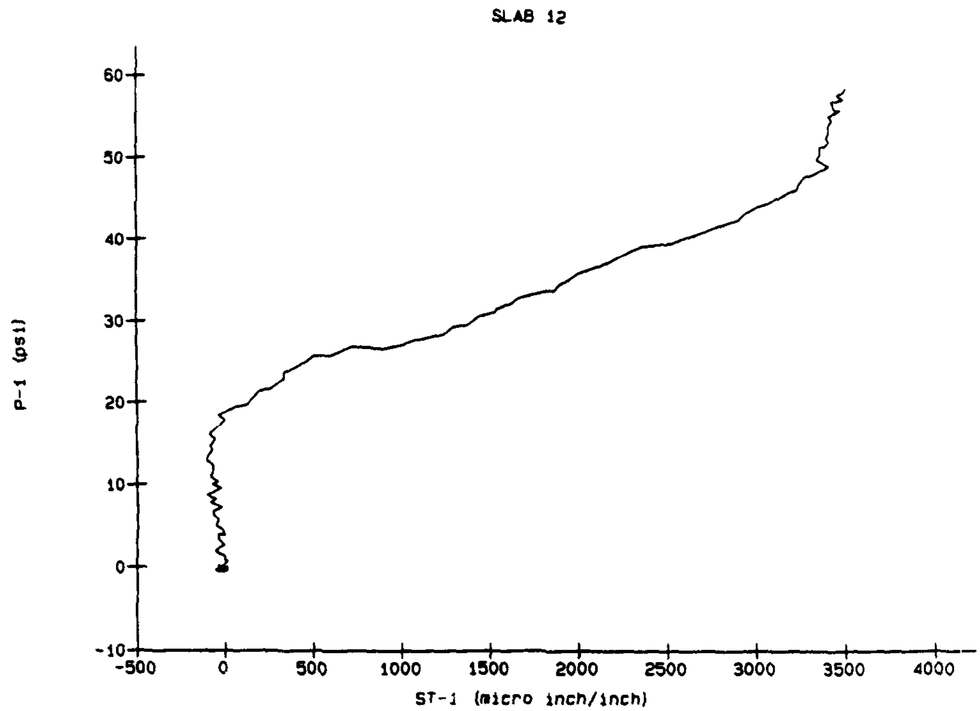


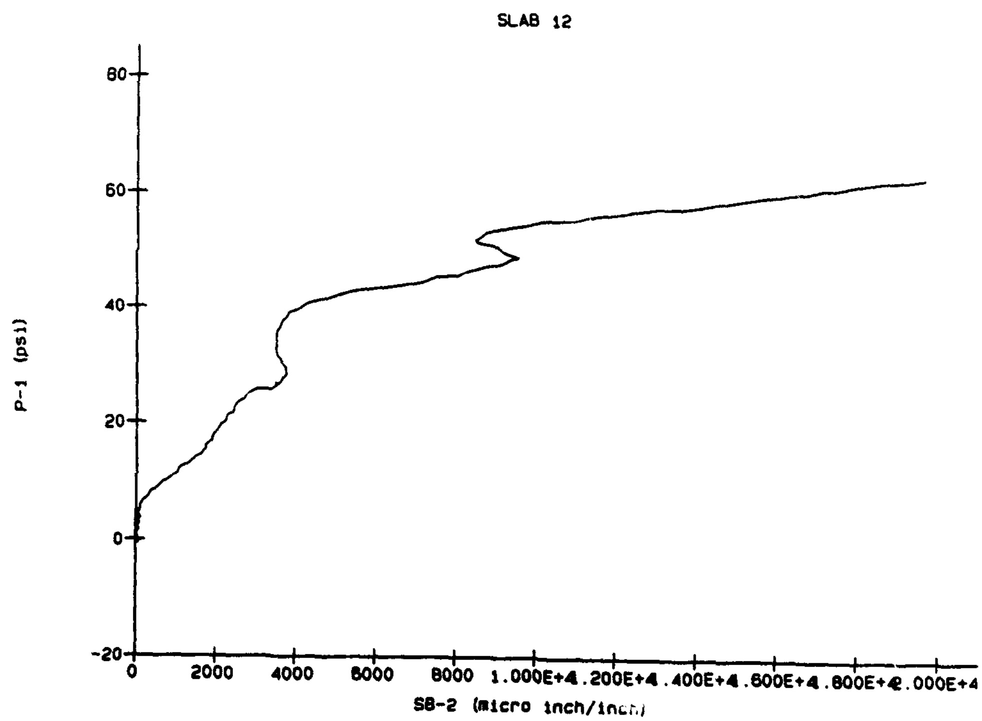
SLAB 12

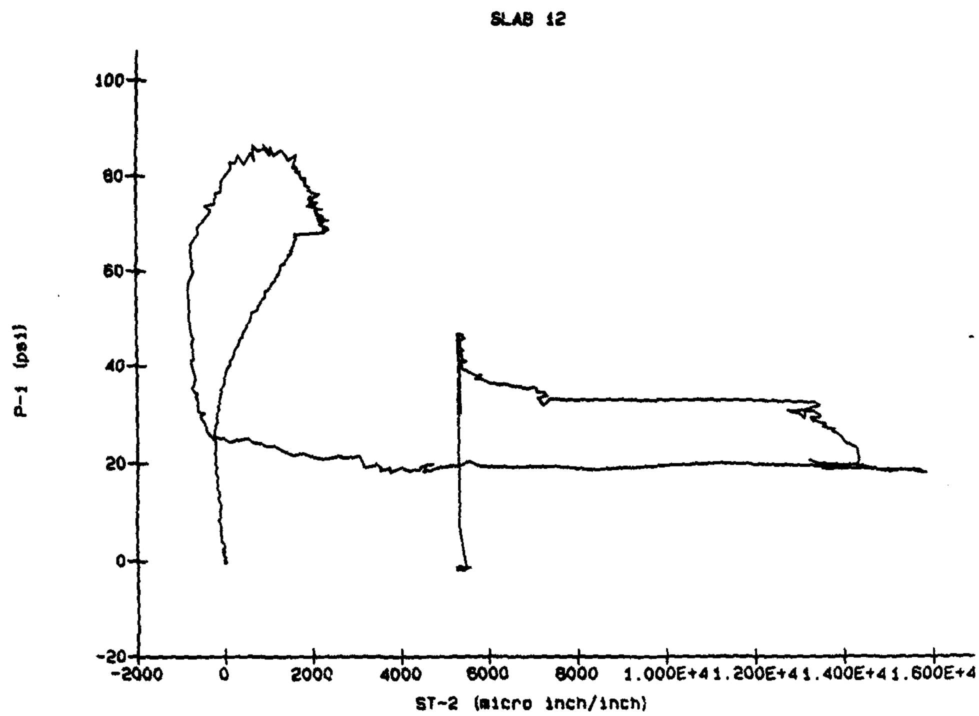


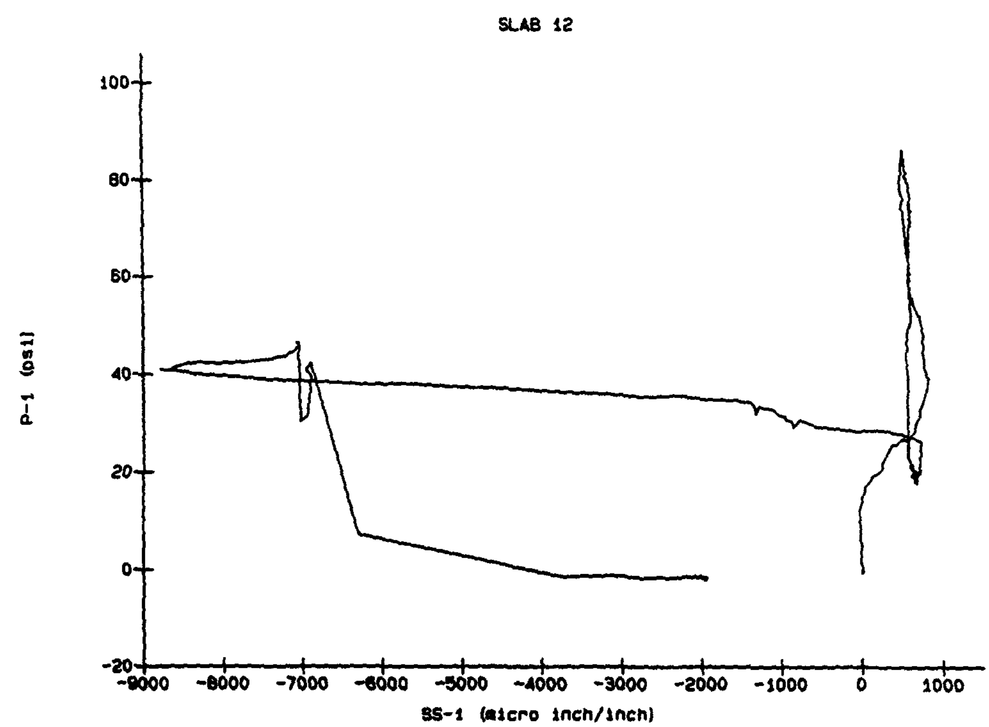

SLAB 12
P-1 (psi)
100
80
60
40
20
0
-20
-0.50
0.0
0.50
1.0
1.5
2.0
2.5
3.0
D-2 (inches)

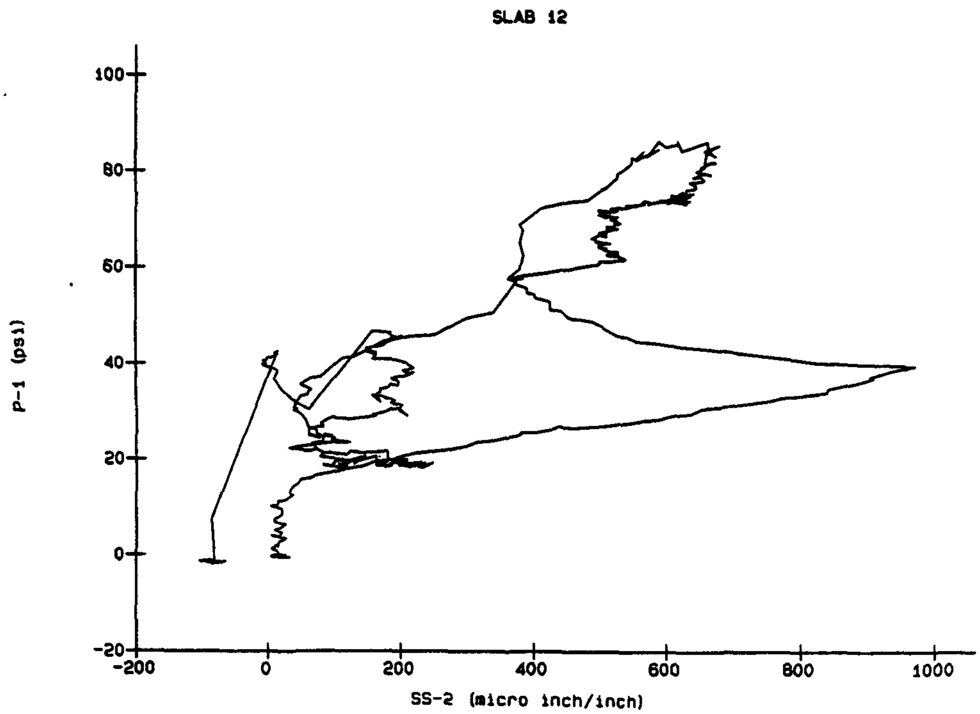

SLAB 12
P-1 (psi)
100
80
60
40
20
0
-20
-500
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
SB-1 (micro inch/inch)

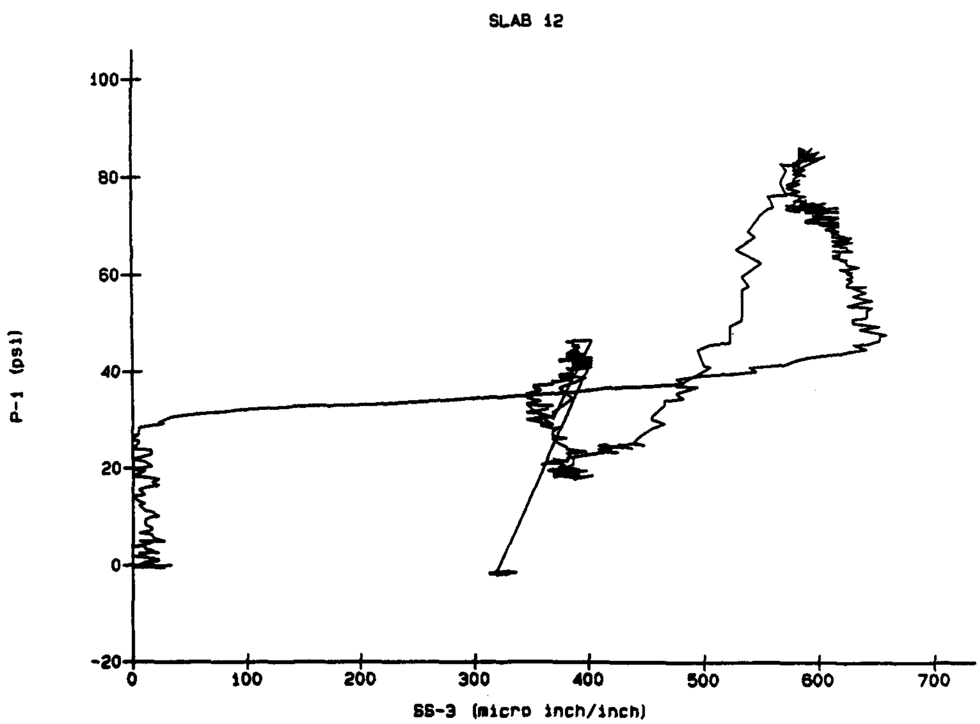

SLAB 12
60
50
40
30
20
10
0
-10
P-1 (psi)
-500
0
500
1000
1500
2000
2500
3000
3500
4000
ST-1 (micro inch/inch)


SLAB 12
80
60
40
20
0
-20
P-1 (psi)
0
2000
4000
6000
8000
1.000E+4
.200E+4
.400E+4
.600E+4
.800E+4
2.000E+4
SB-2 (micro inch/inch)

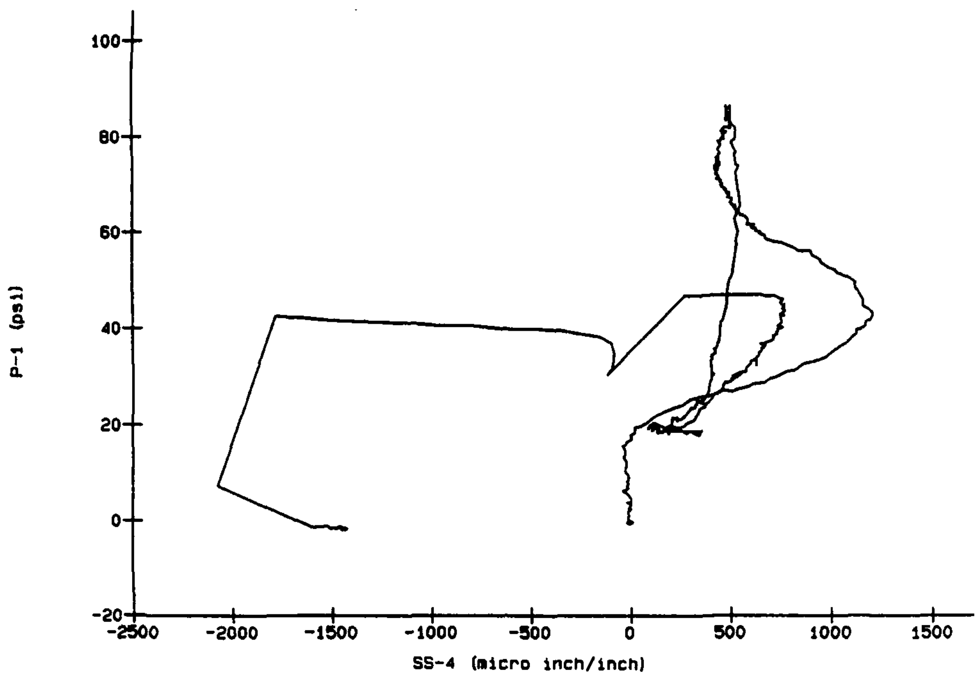

SLAB 12
P-1 (psi)
100
80
60
40
20
0
-20
-2000
0
2000
4000
6000
8000
1.000E+4
1.200E+4
1.400E+4
1.600E+4
ST-2 (micro inch/inch)


SLAB 12
P-1 (psi)
100
80
60
40
20
0
-20
-9000
-8000
-7000
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
SS-1 (micro inch/inch)

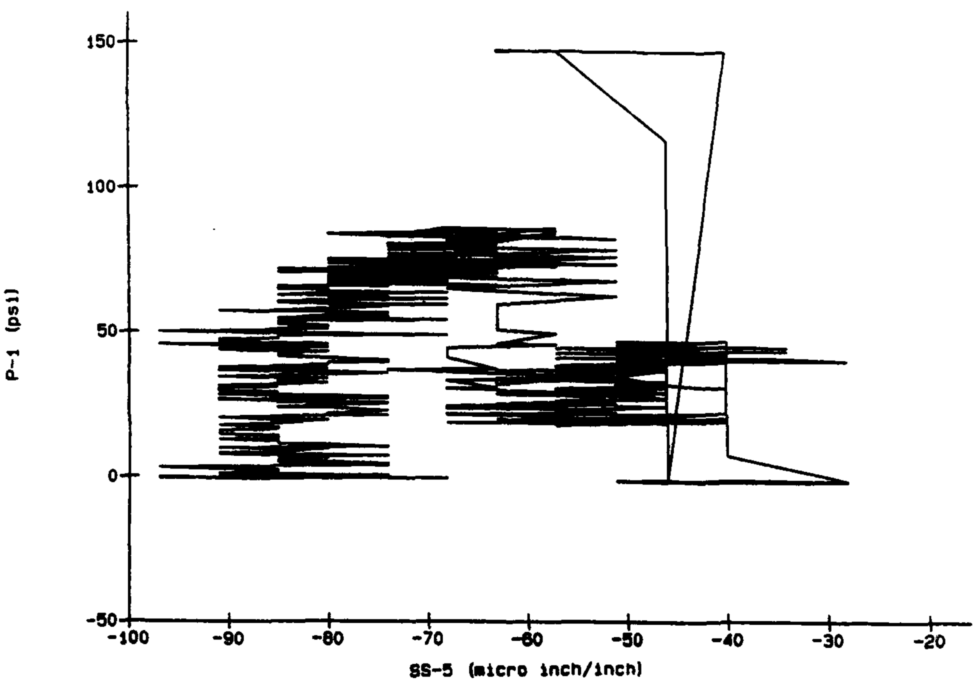

SLAB 12
P-1 (psi)
SS-2 (micro inch/inch)
100
80
60
40
20
0
-20
-200
0
200
400
600
800
1000


SLAB 12
P-1 (psi)
SS-3 (micro inch/inch)
100
80
60
40
20
0
-20
0
100
200
300
400
500
600
700

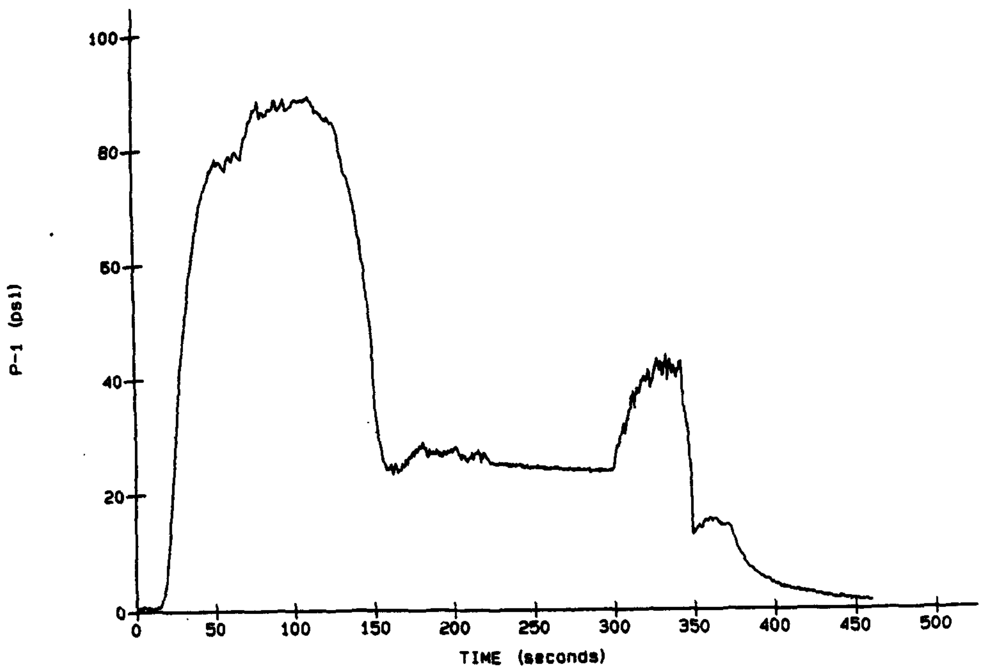
SLAB 12



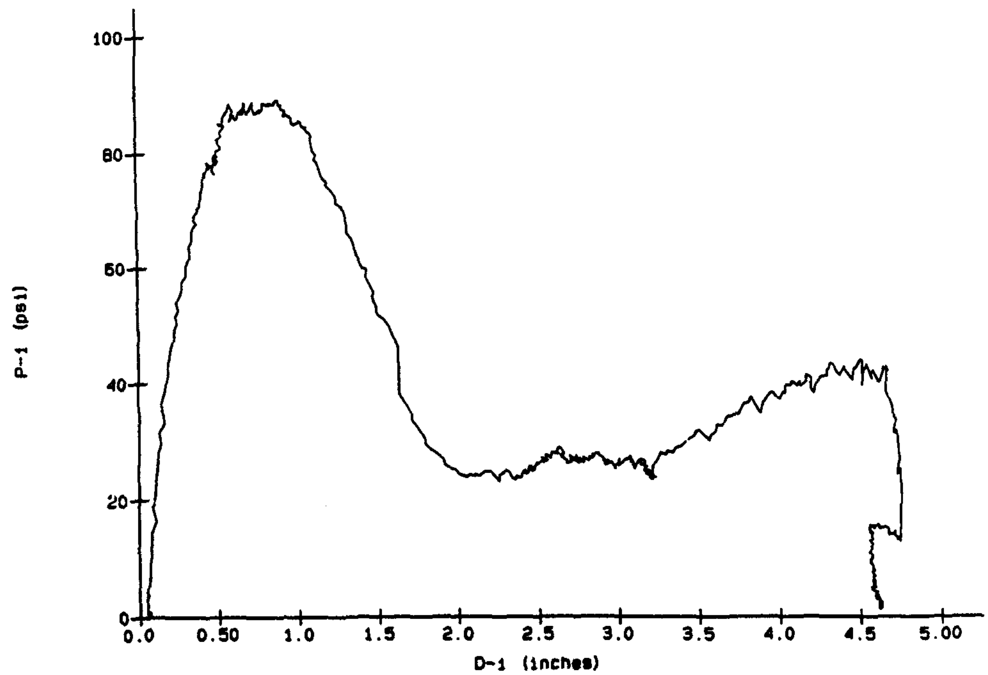
SLAB 12



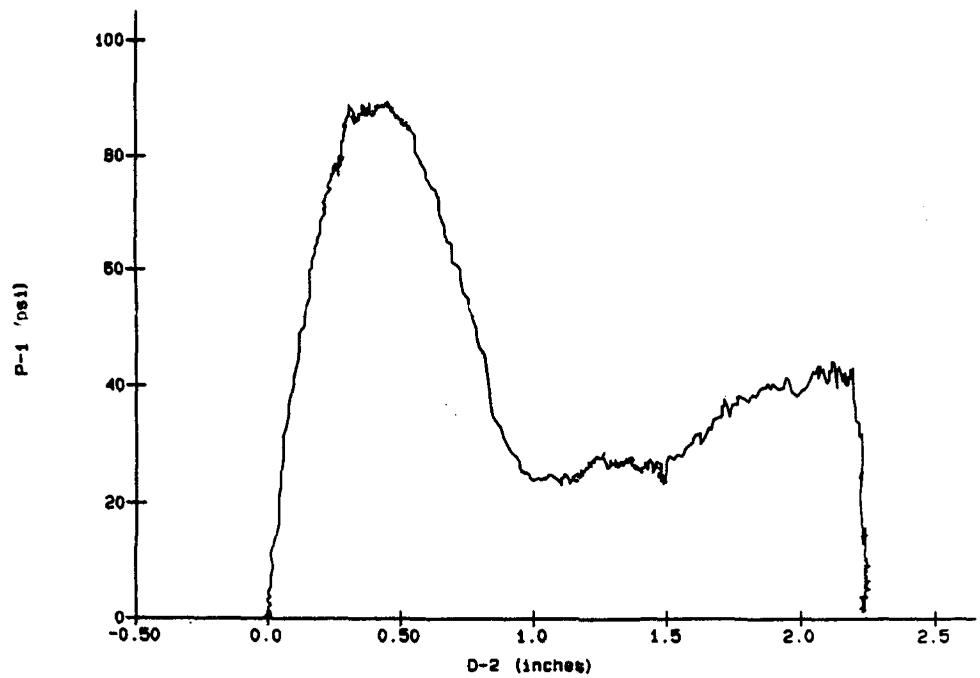
SLAB 13



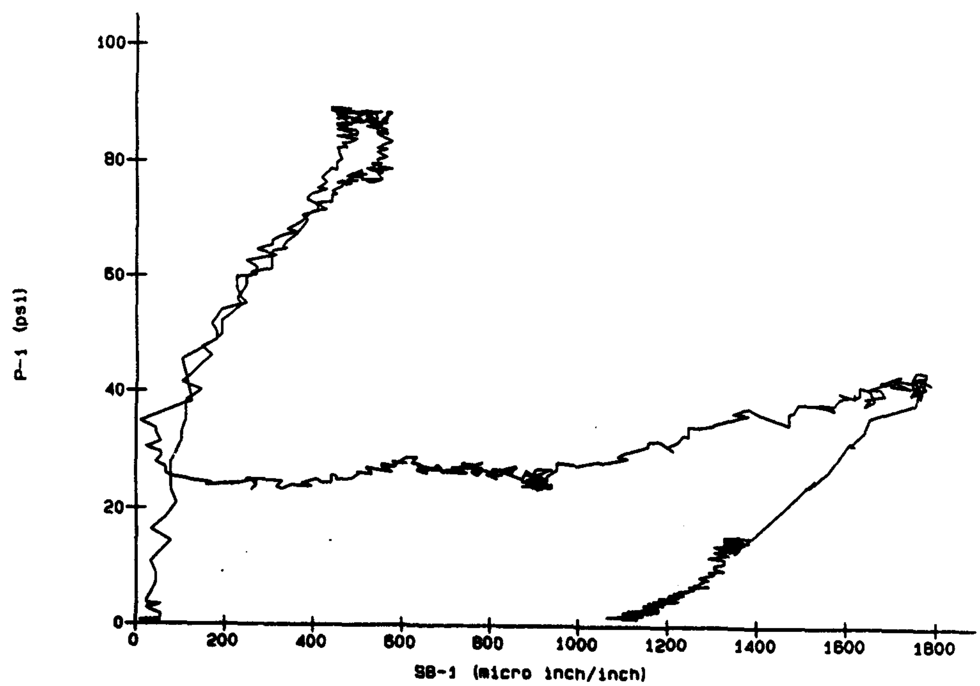
SLAB 13



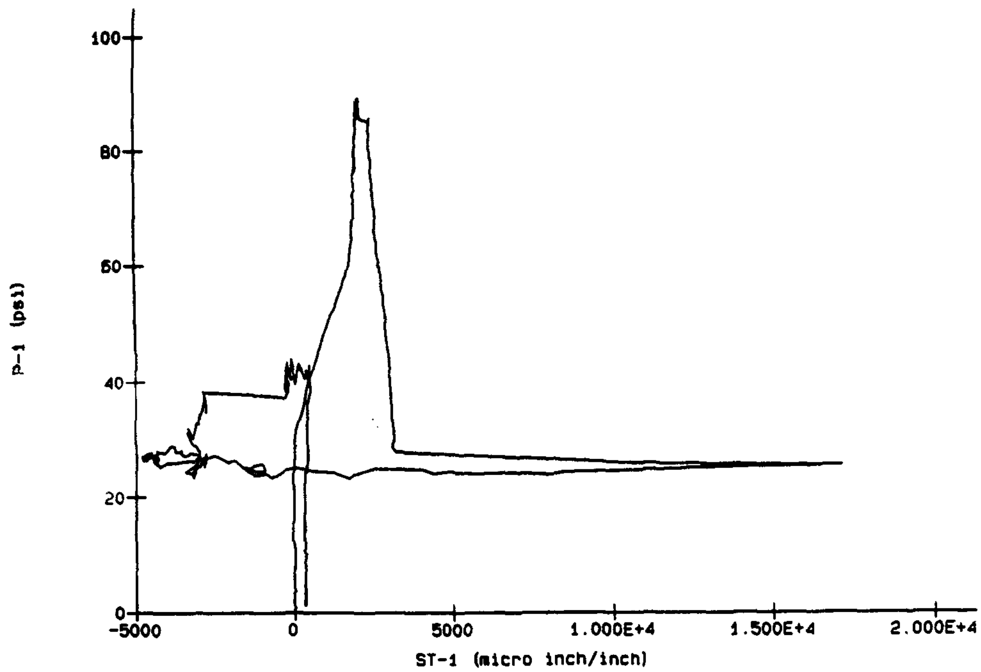
SLAB 13



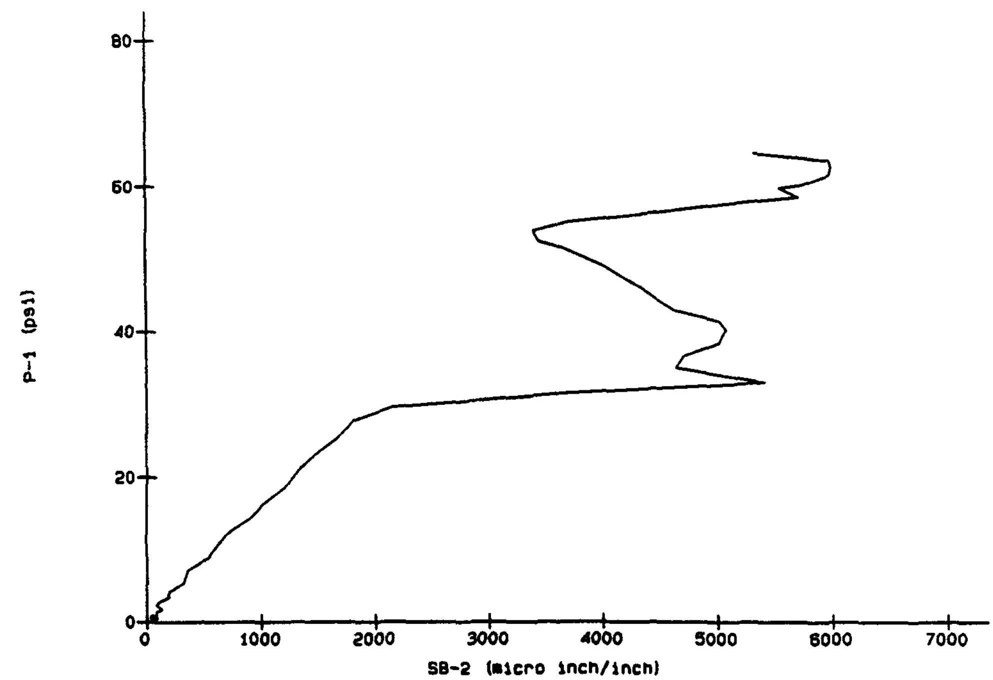
SLAB 13



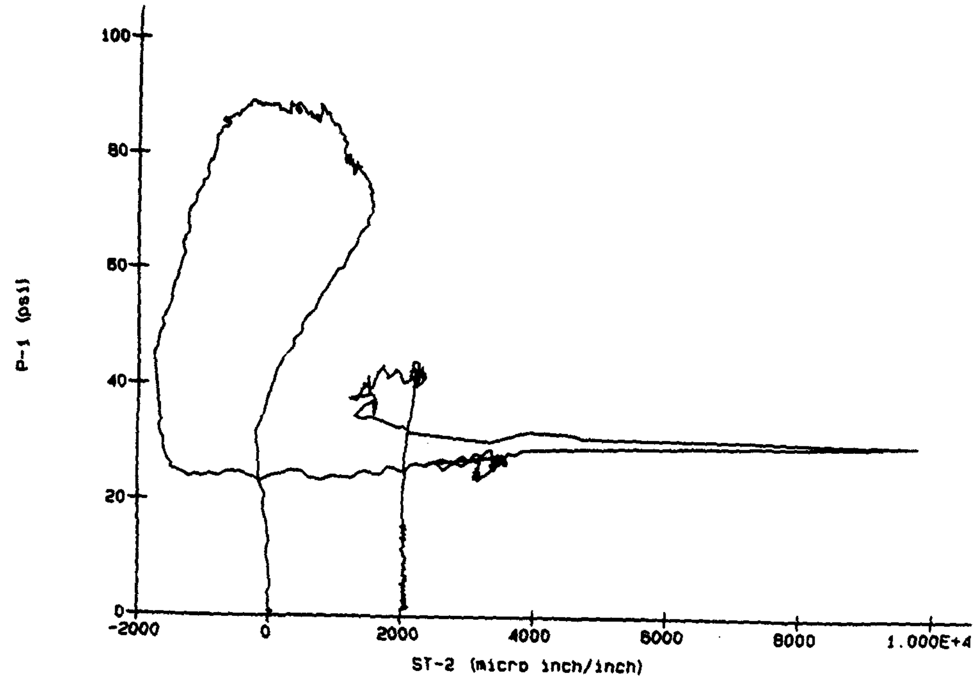
SLAB 13



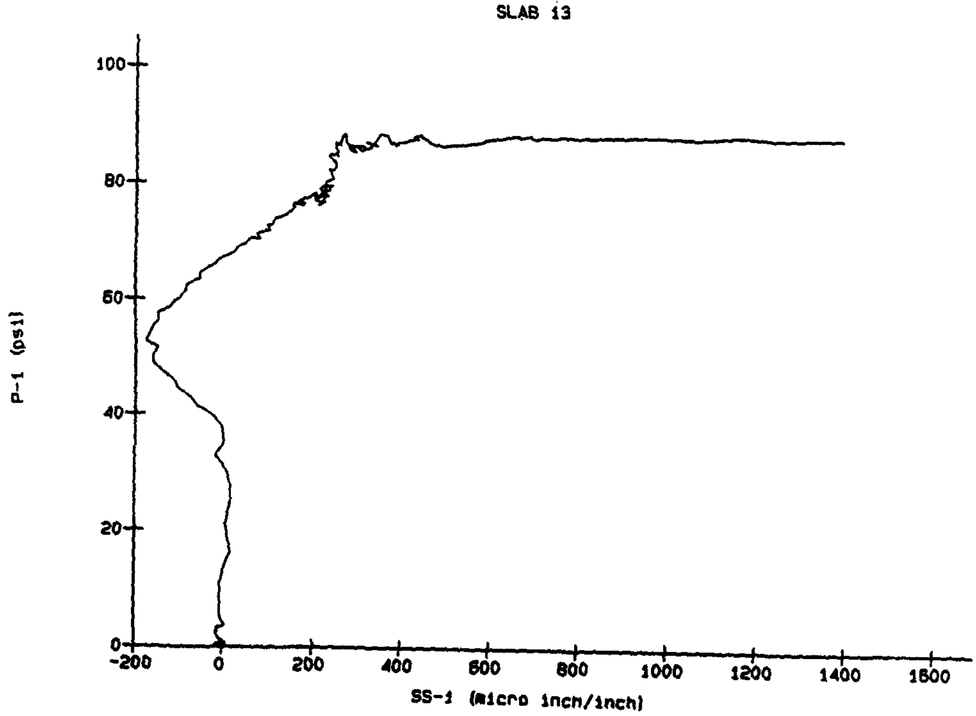
SLAB 13

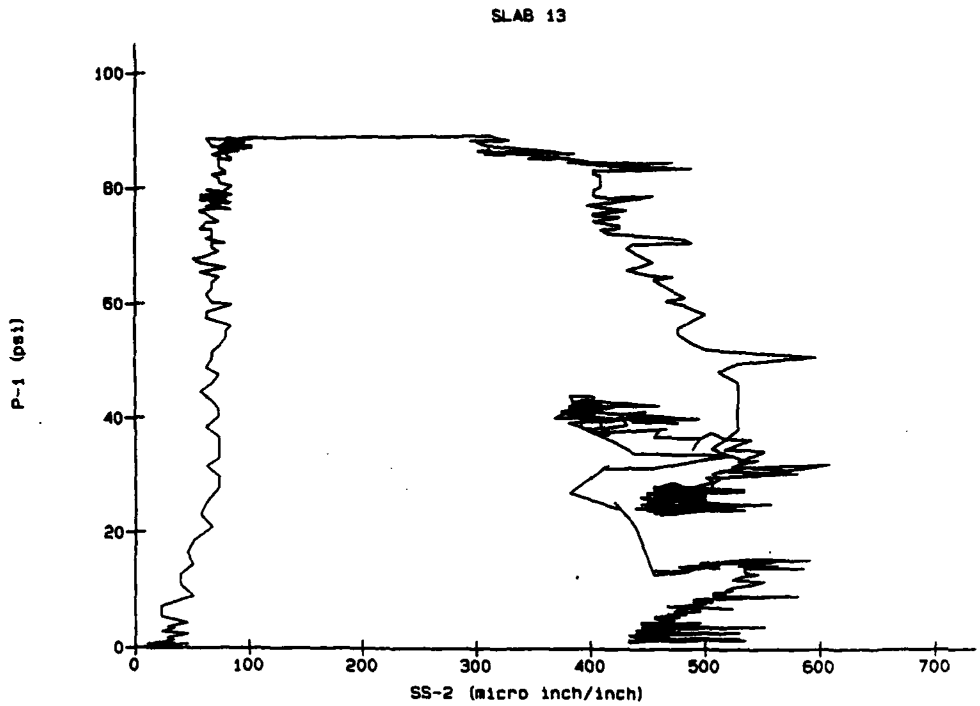


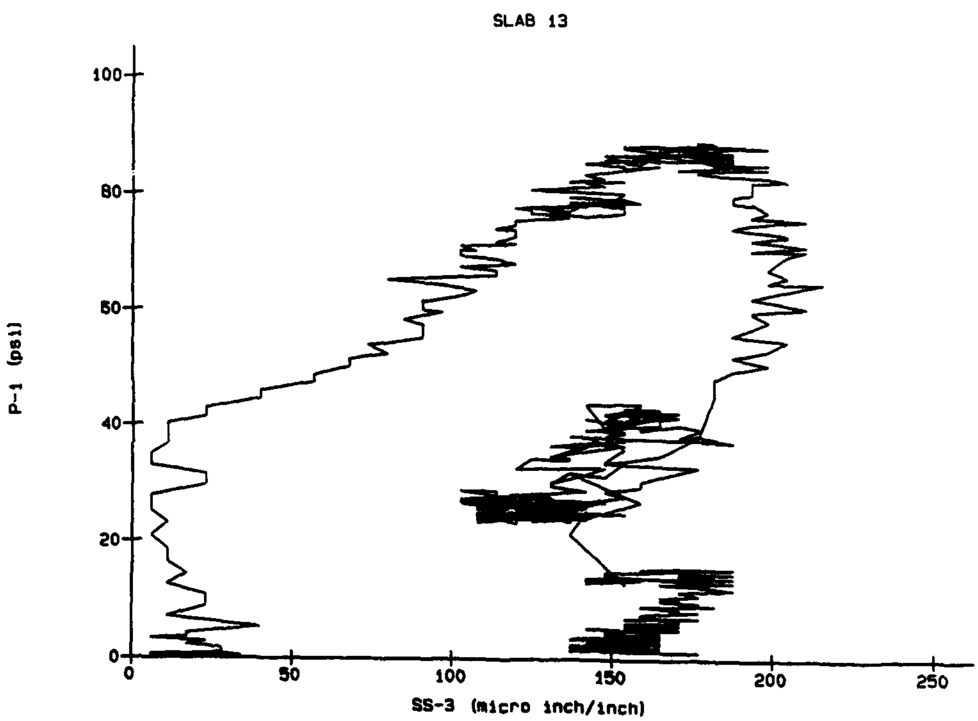
SLAB 13

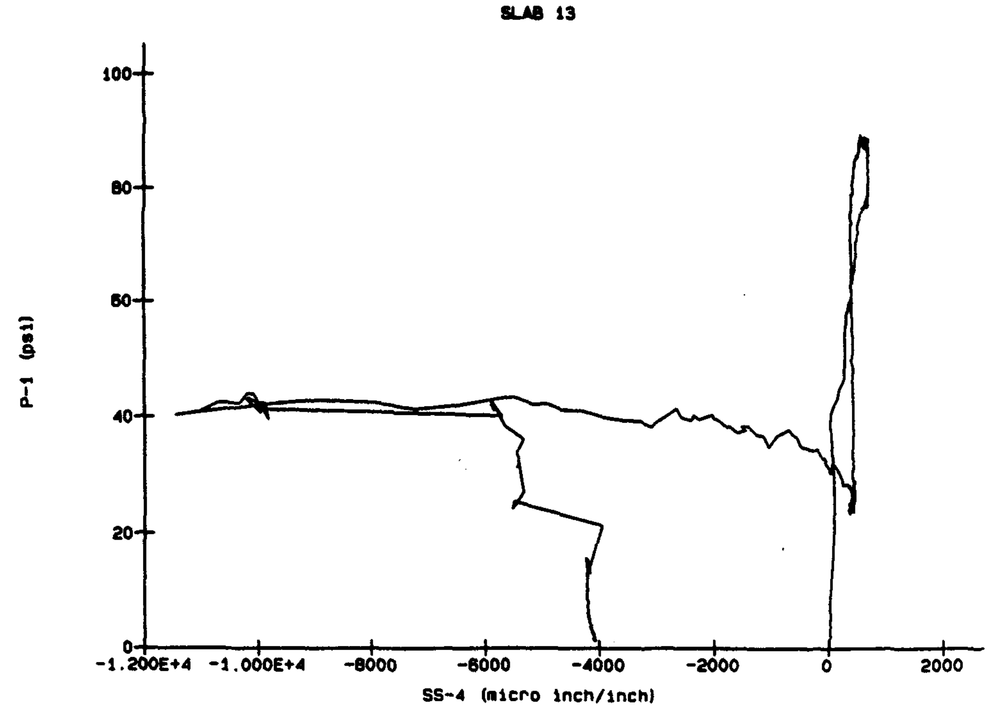


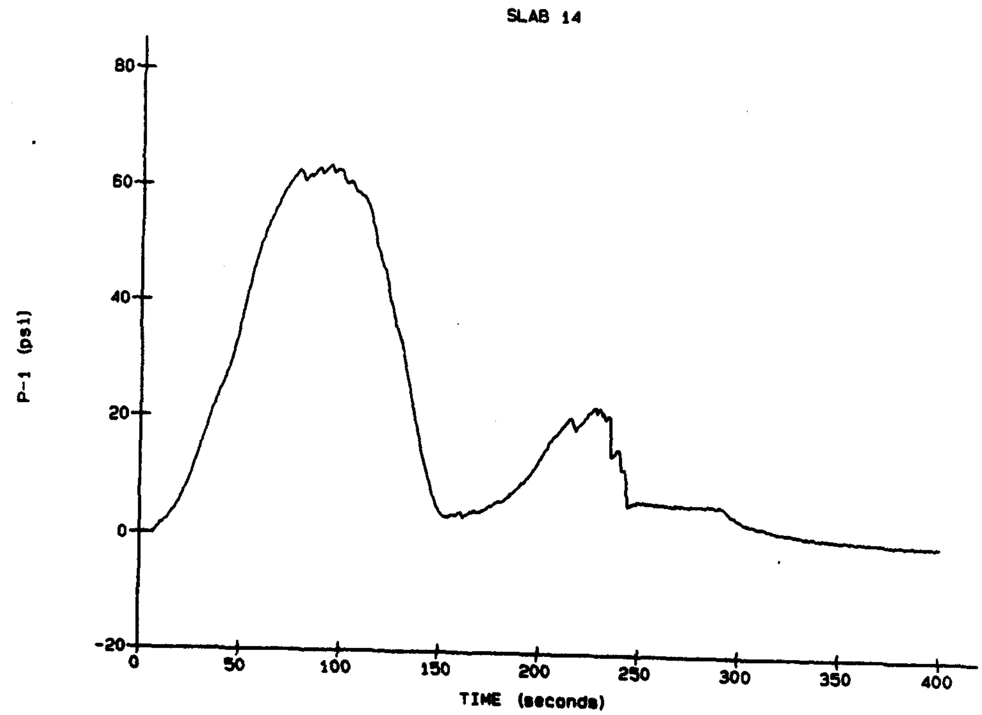
SLAB 13

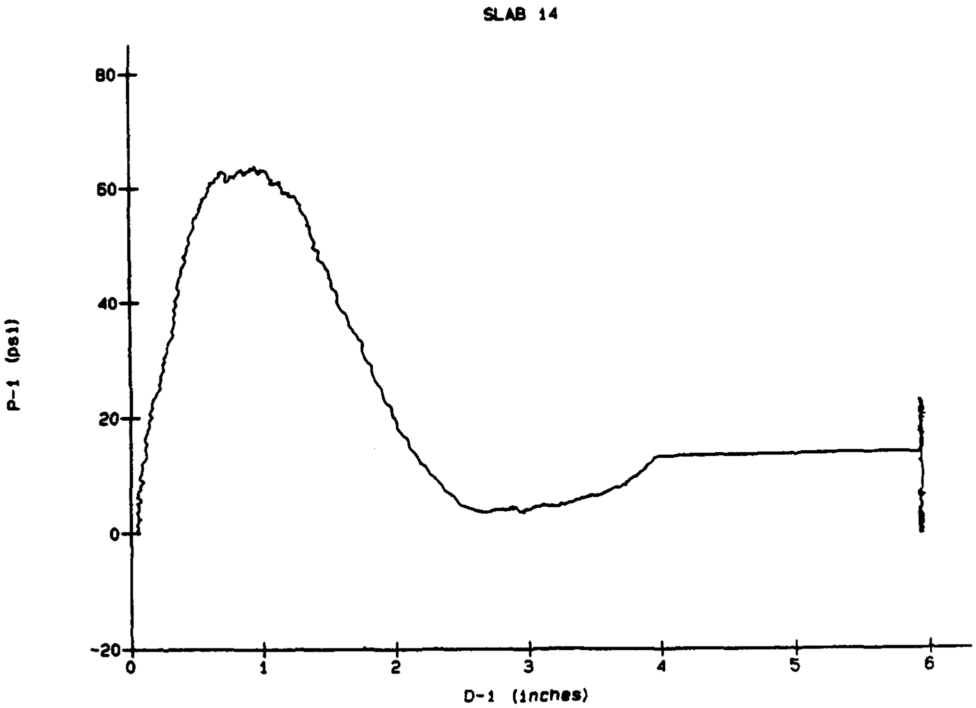


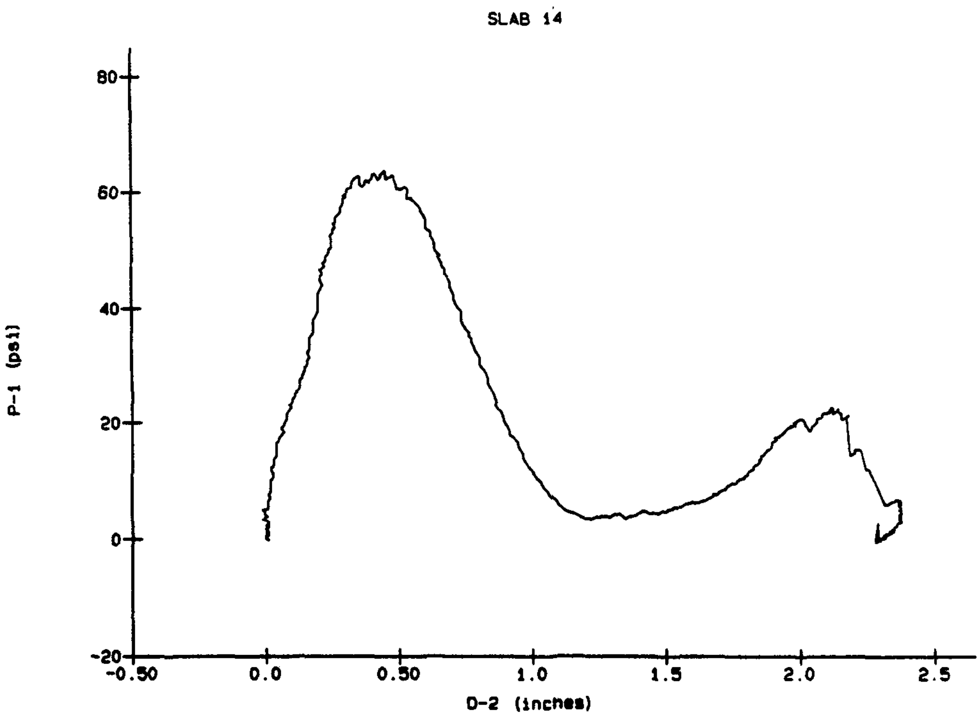

SLAB 13
P-1 (psi)
0
20
40
60
80
100
0
100
200
300
400
500
600
700
SS-2 (micro inch/inch)


SLAB 13
P-1 (psi)
0
20
40
60
80
100
0
50
100
150
200
250
SS-3 (micro inch/inch)

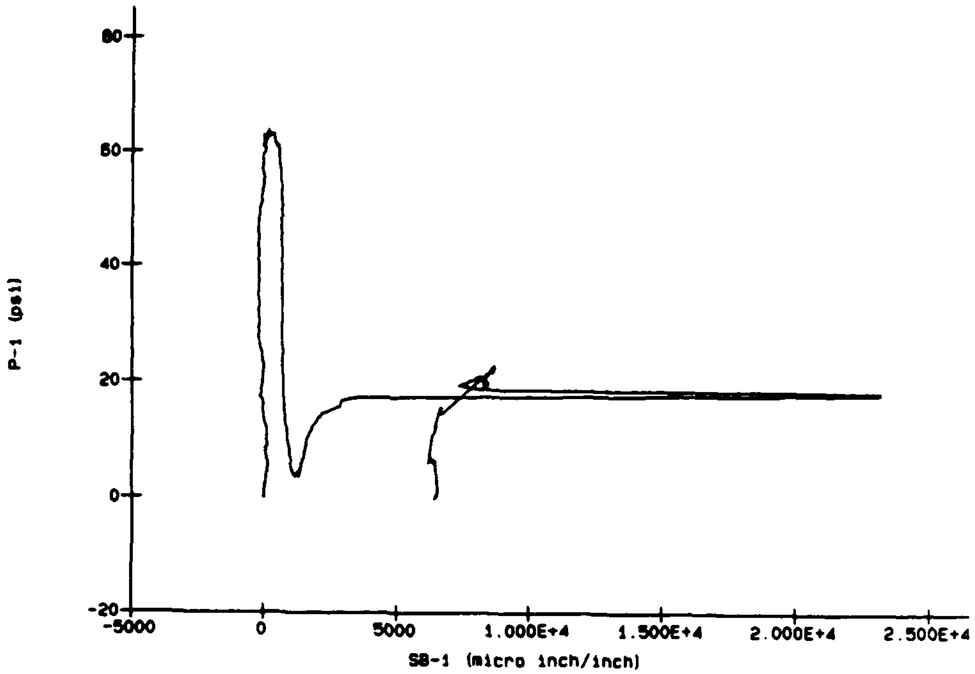

SLAB 13
P-1 (psi)
100
80
60
40
20
0
-1.200E+4
-1.000E+4
-8000
-6000
-4000
-2000
0
2000
SS-4 (micro inch/inch)


SLAB 14
P-1 (psi)
80
60
40
20
0
-20
0
50
100
150
200
250
300
350
400
TIME (seconds)

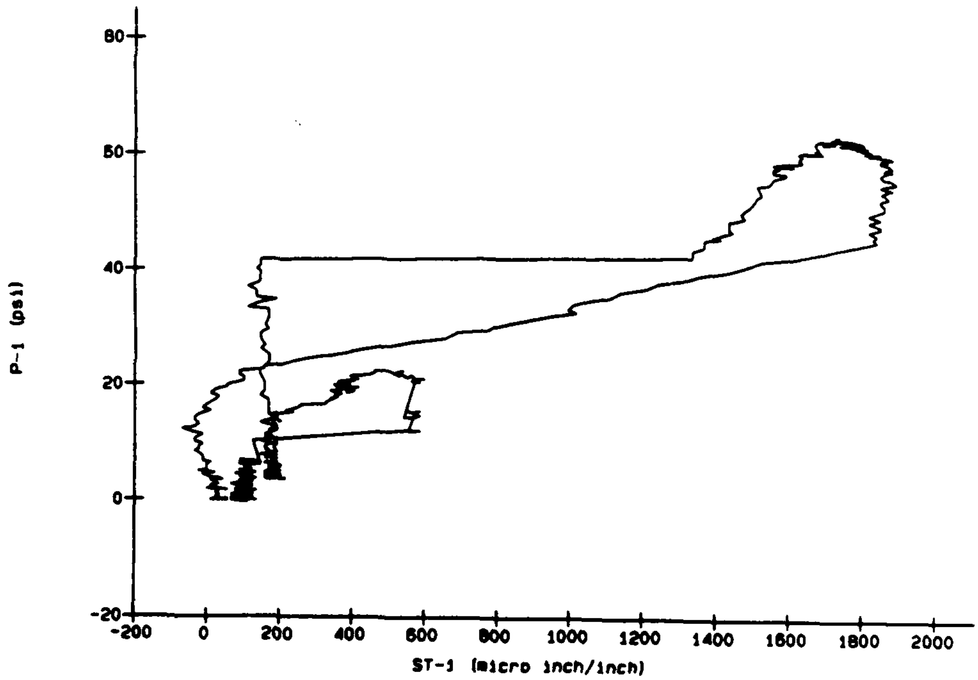

SLAB 14
80
60
40
20
0
-20
P-1 (psi)
0
1
2
3
4
5
6
D-1 (inches)

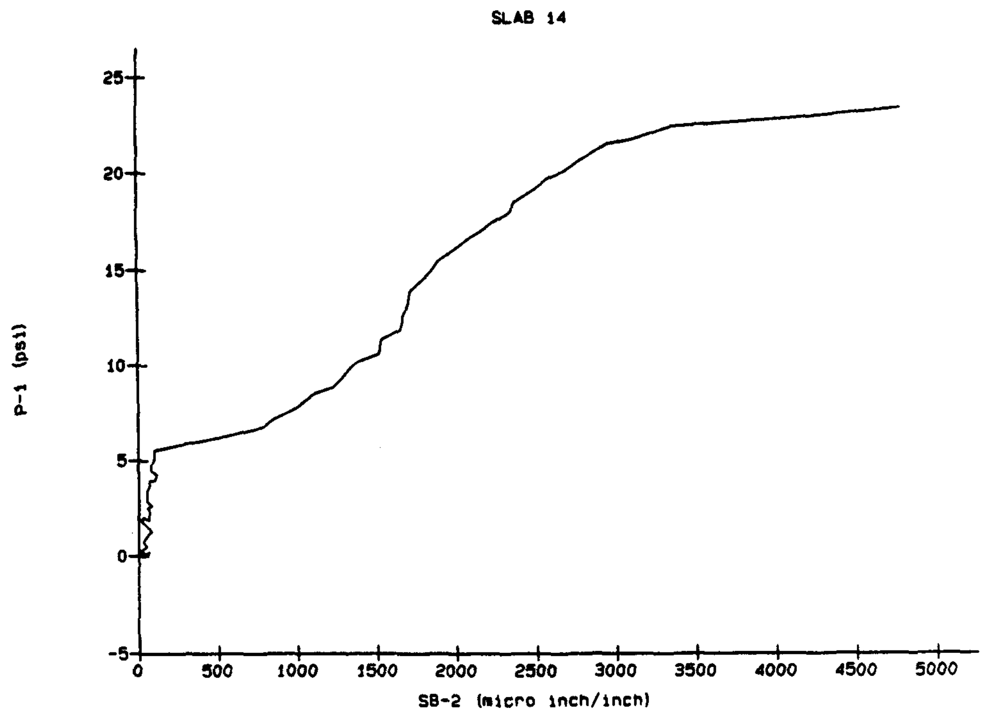

SLAB 14
80
60
40
20
0
-20
P-1 (psi)
-0.50
0.0
0.50
1.0
1.5
2.0
2.5
D-2 (inches)

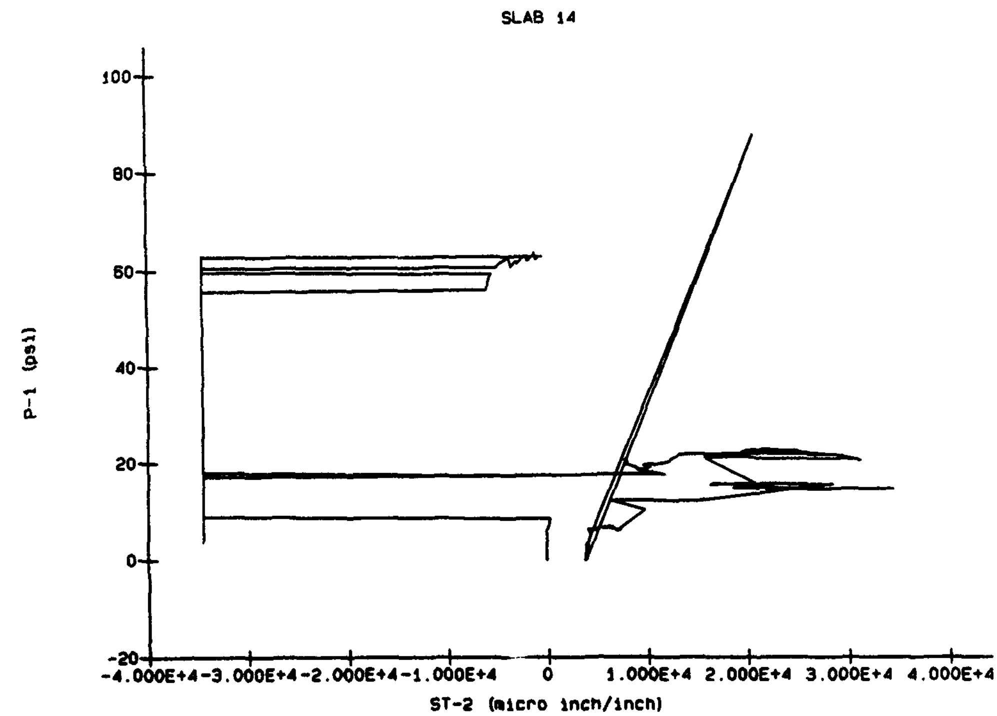
SLAB 14



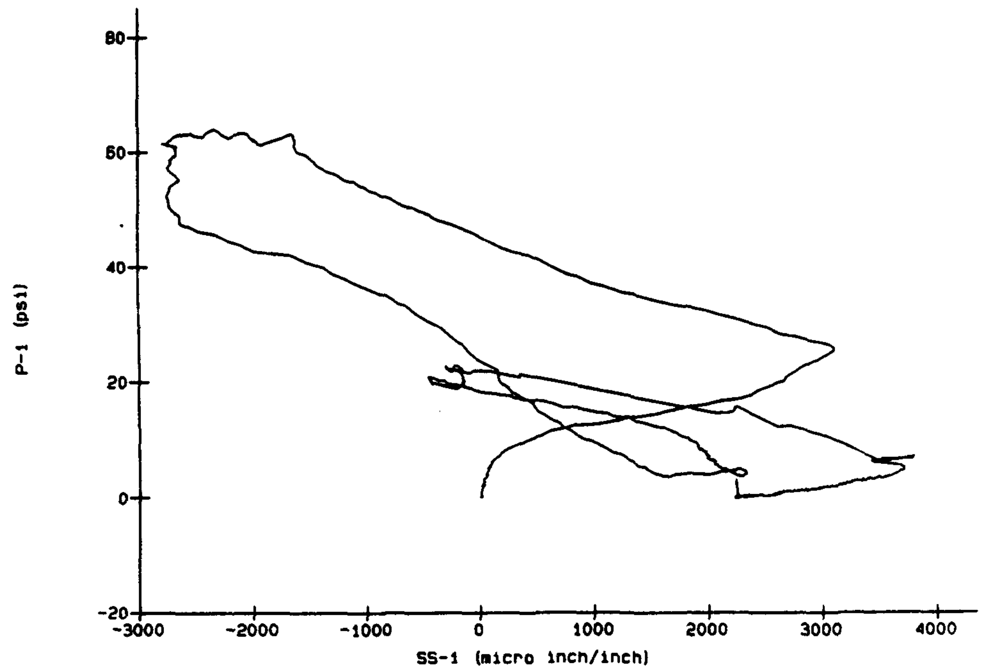
SLAB 14



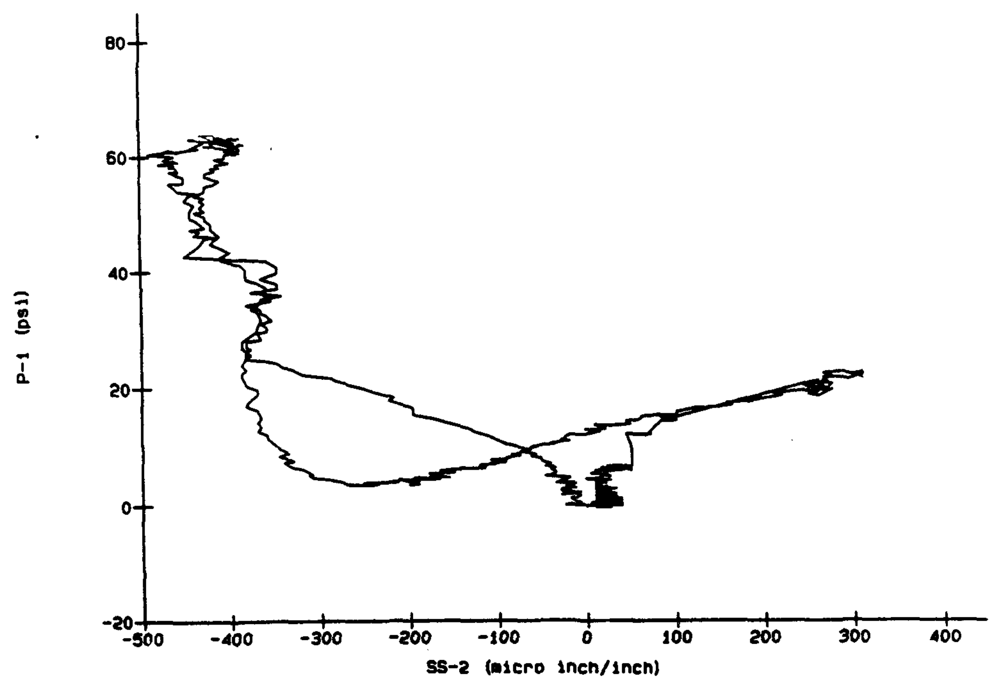

SLAB 14
P-1 (psi)
25
20
15
10
5
0
-5
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
SB-2 (micro inch/inch)

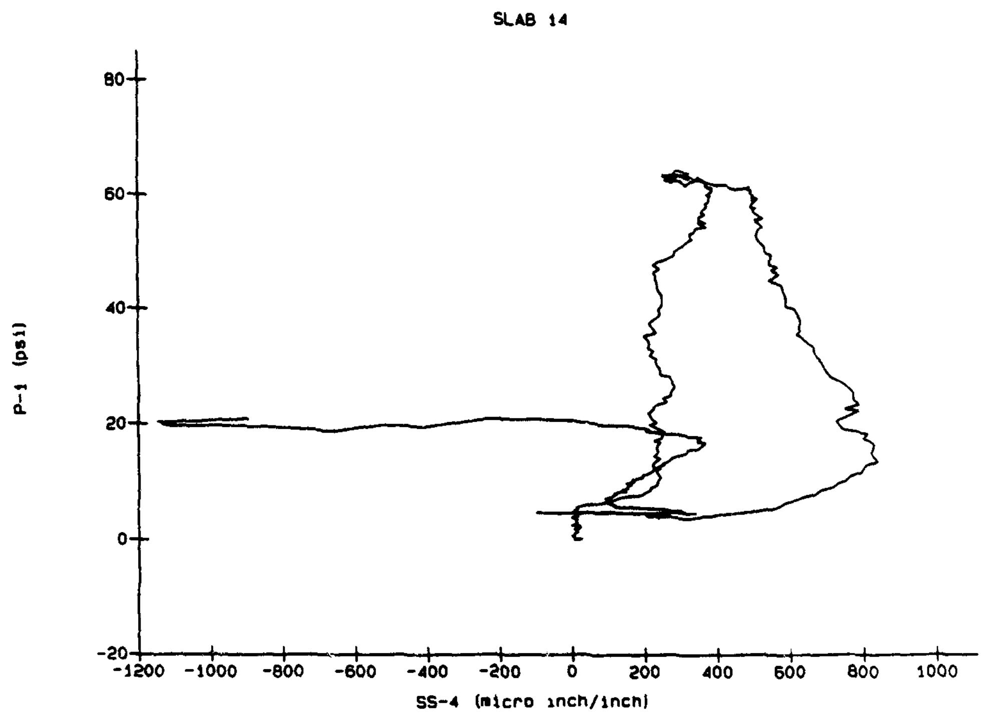

SLAB 14
P-1 (psi)
100
80
60
40
20
0
-20
-4.000E+4
-3.000E+4
-2.000E+4
-1.000E+4
0
1.000E+4
2.000E+4
3.000E+4
4.000E+4
ST-2 (micro inch/inch)

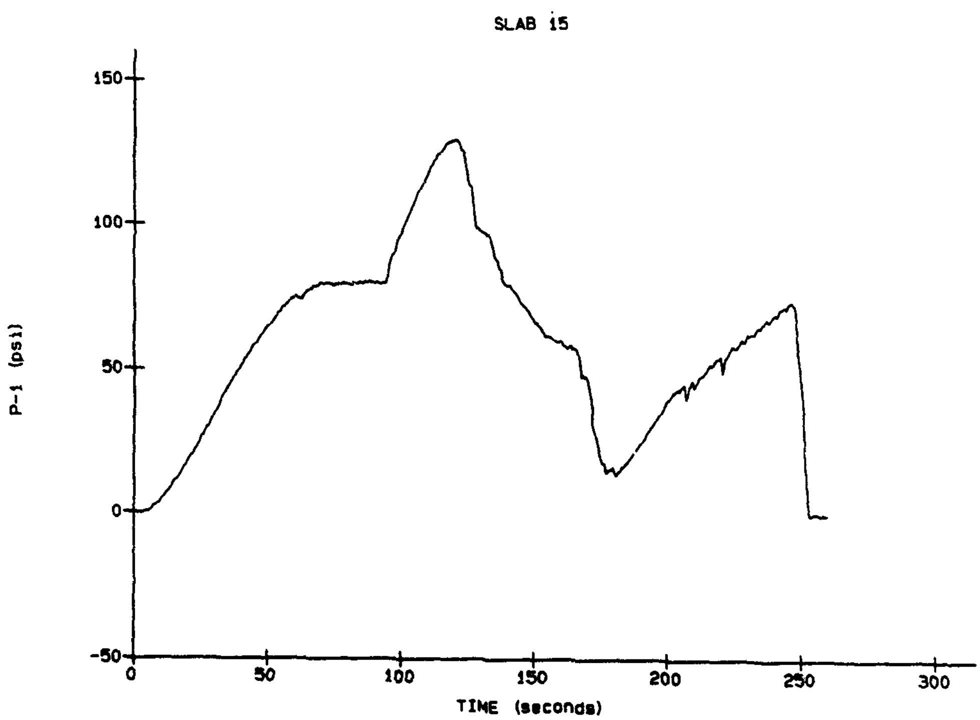
SLAB 14



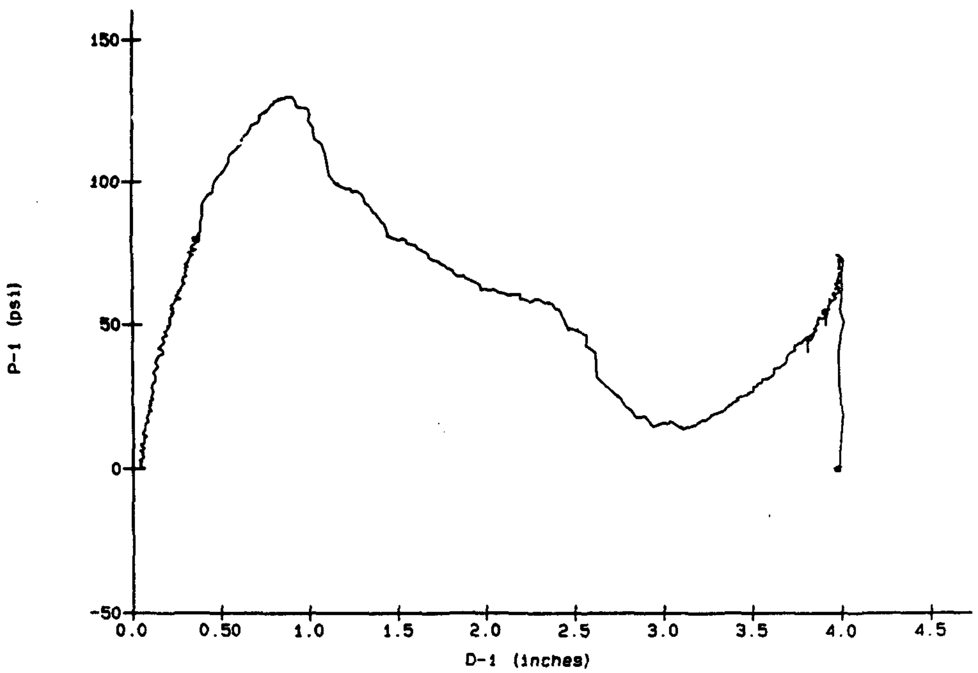
SLAB 14



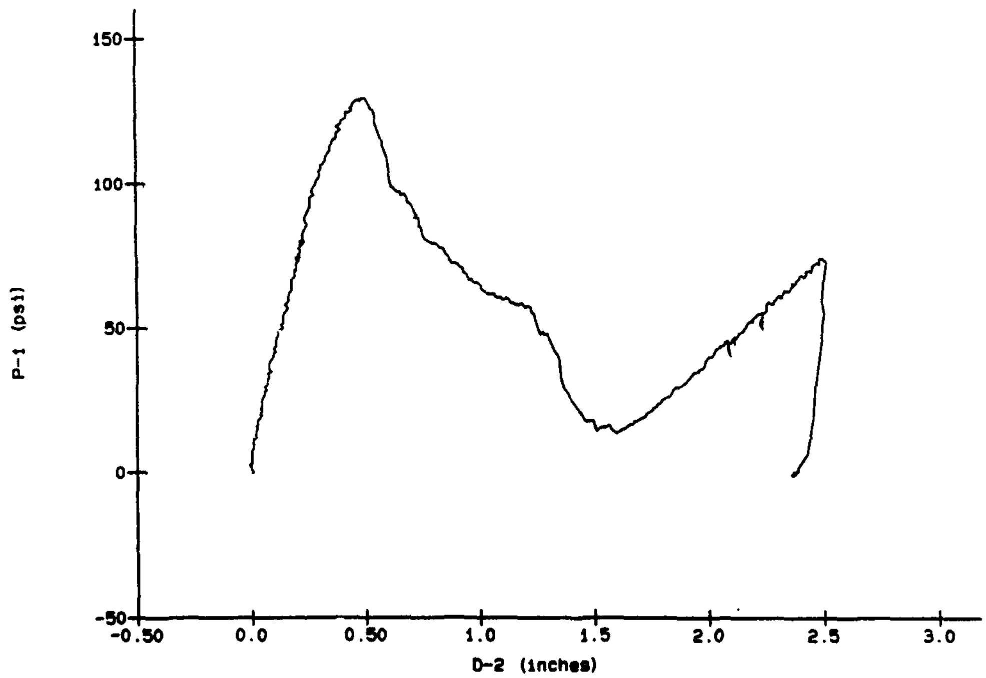

SLAB 14
80
60
40
20
0
-20
P-1 (psi)
-1200
-1000
-800
-600
-400
-200
0
200
400
600
800
1000
SS-4 (micro inch/inch)


SLAB 15
150
100
50
0
-50
P-1 (psi)
0
50
100
150
200
250
300
TIME (seconds)

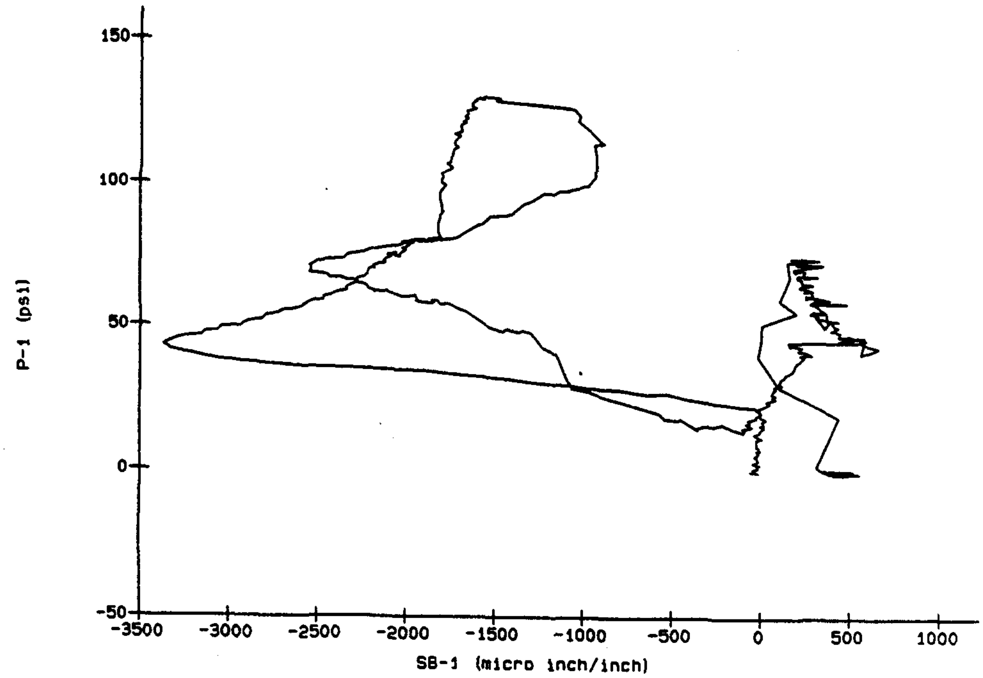
SLAB 15



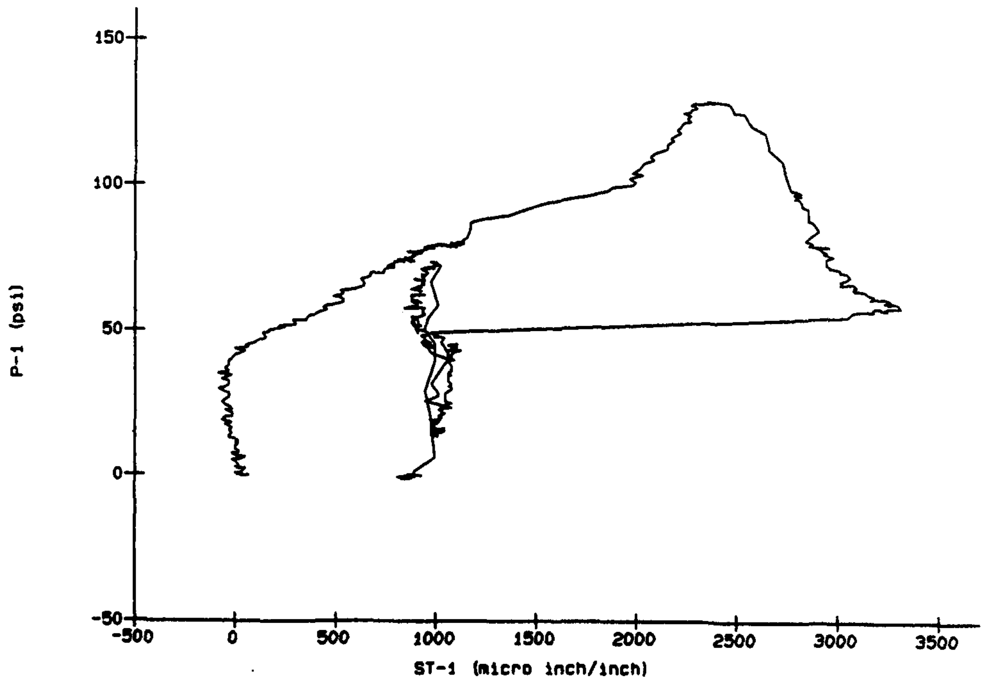
SLAB 15



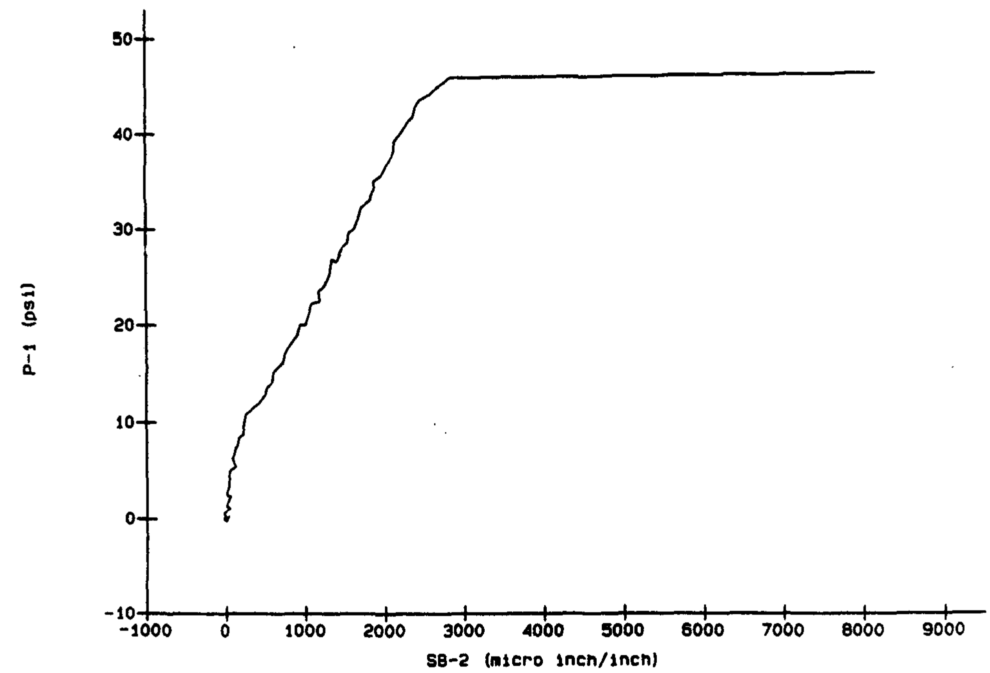
SLAB 15



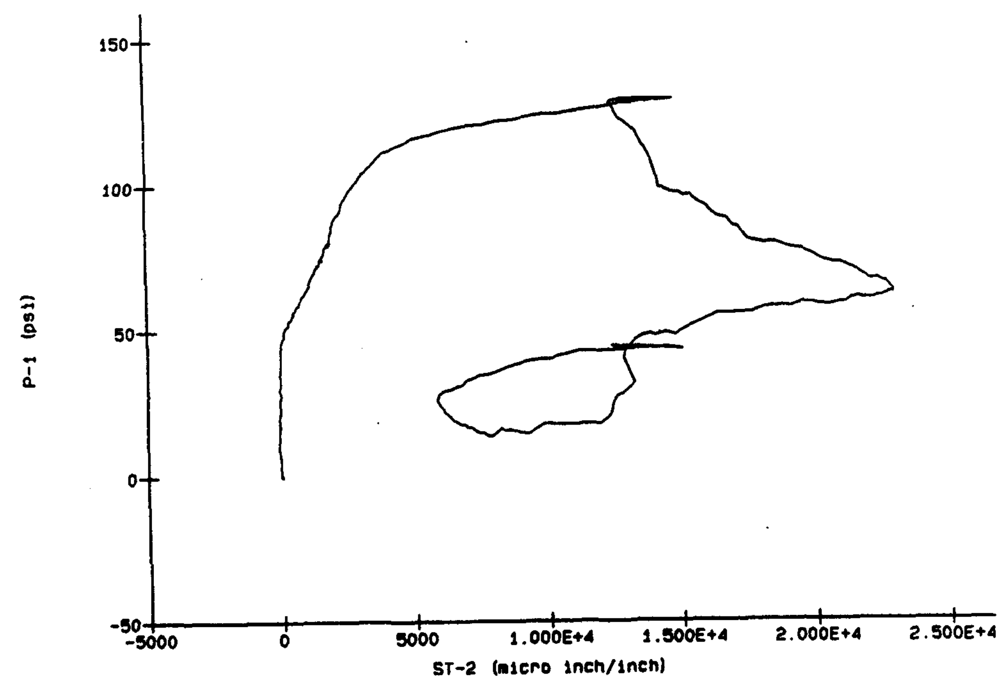
SLAB 15



SLAB 15



SLAB 15



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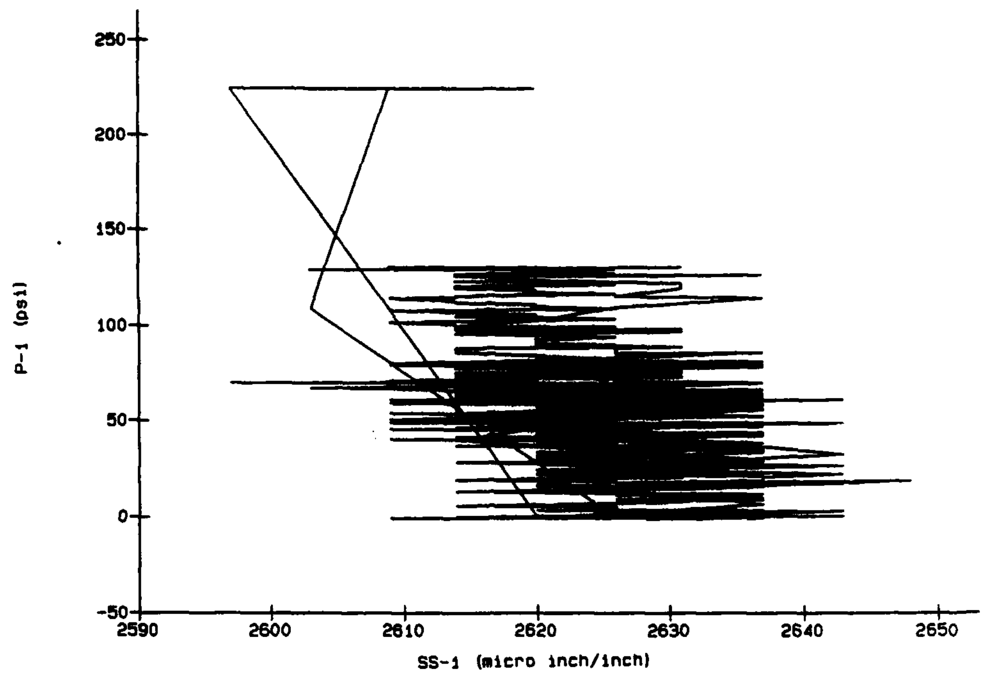

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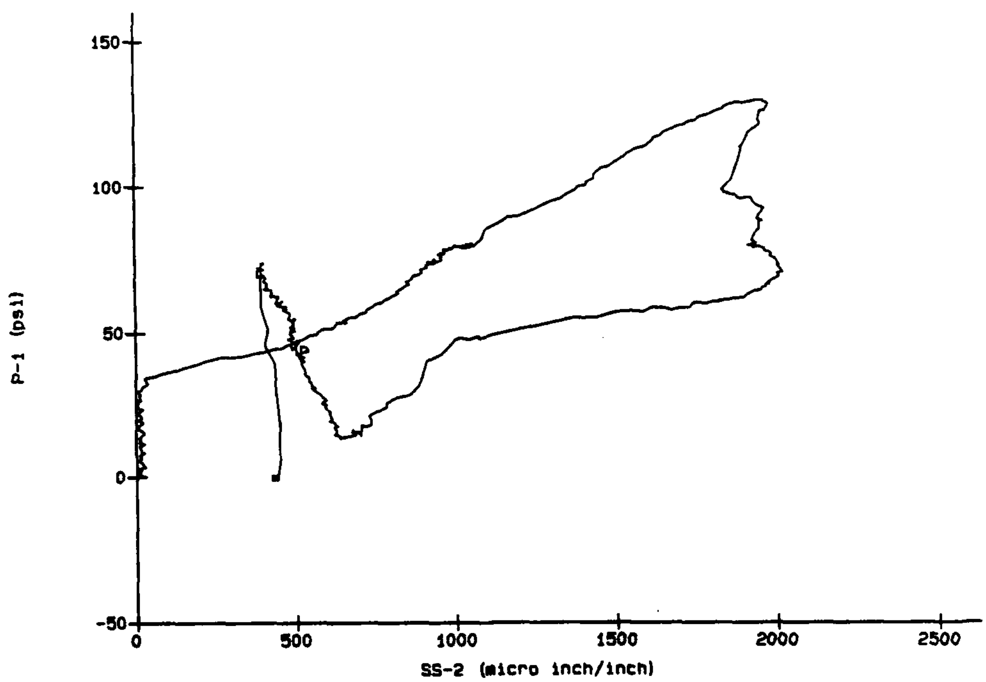
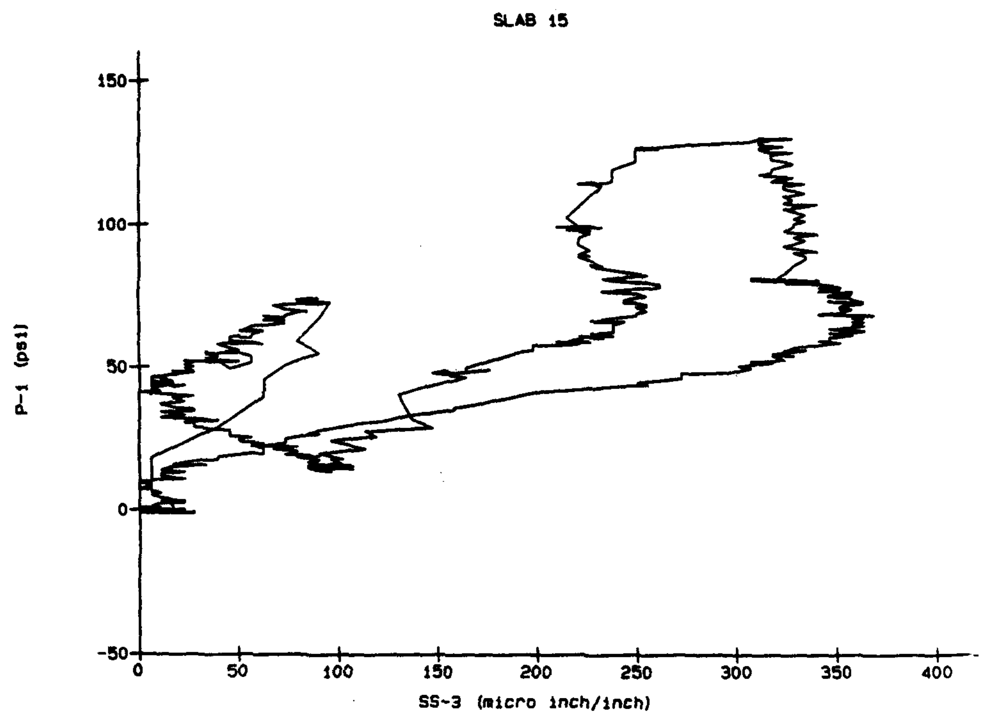
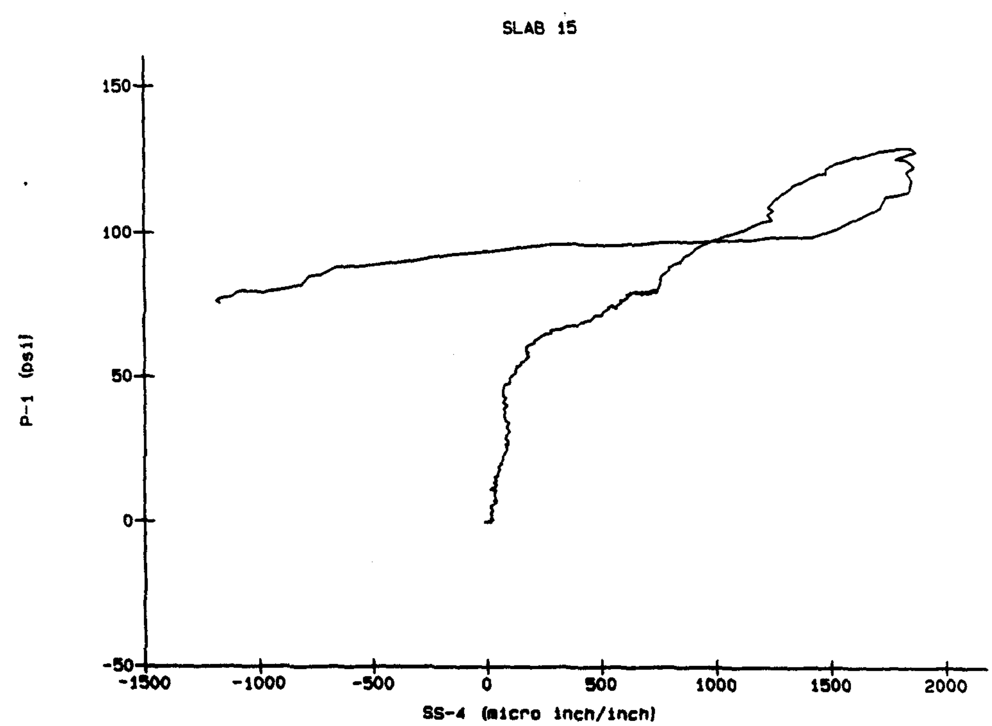
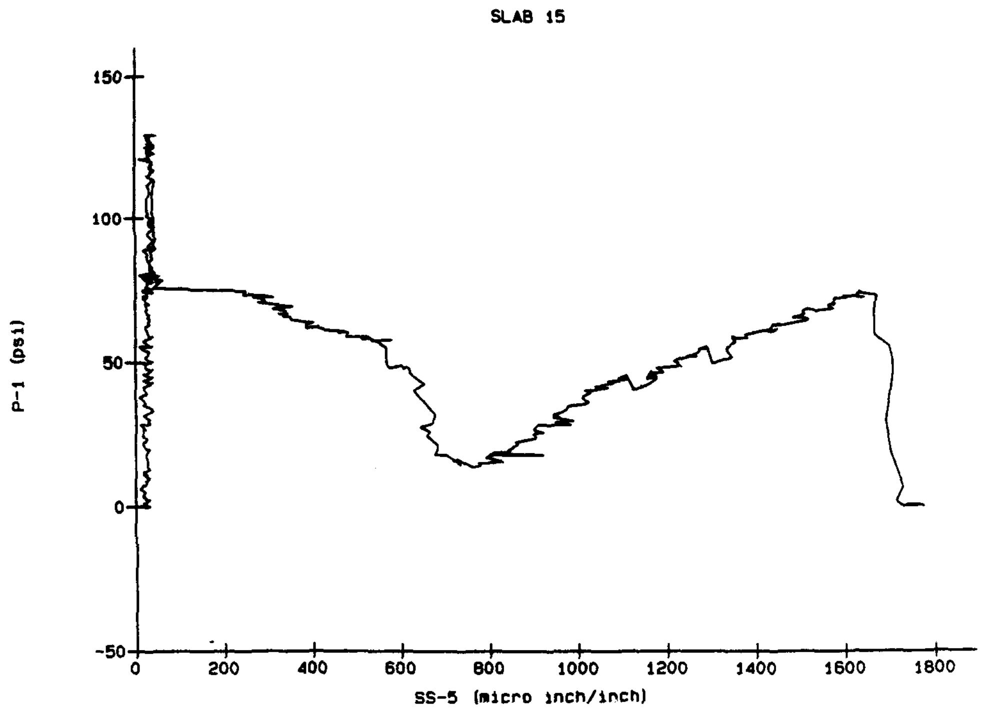
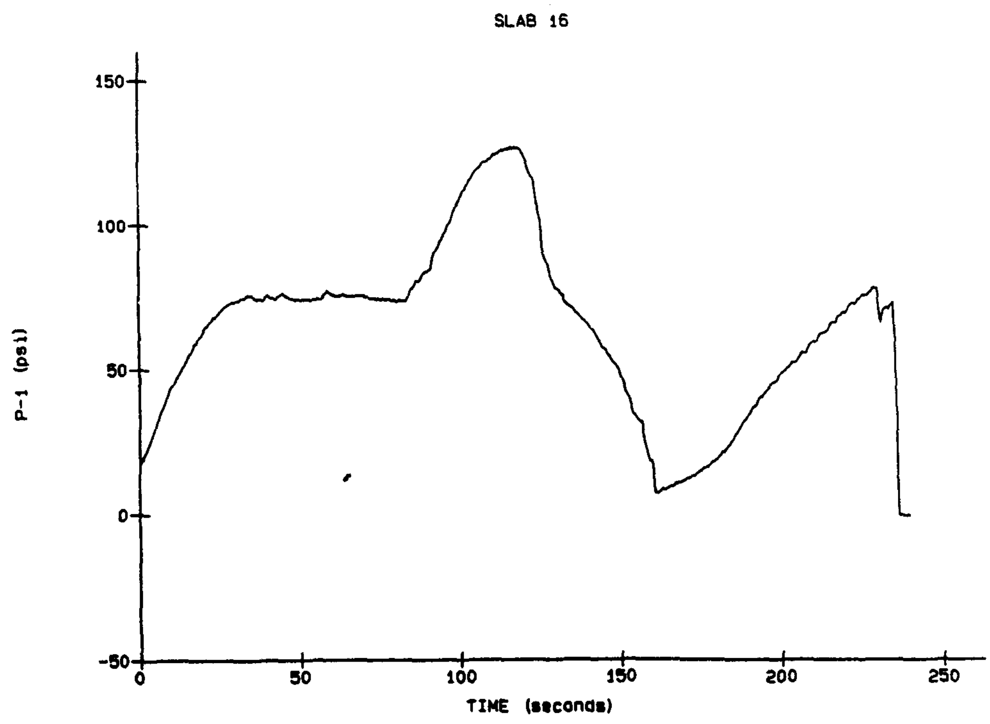


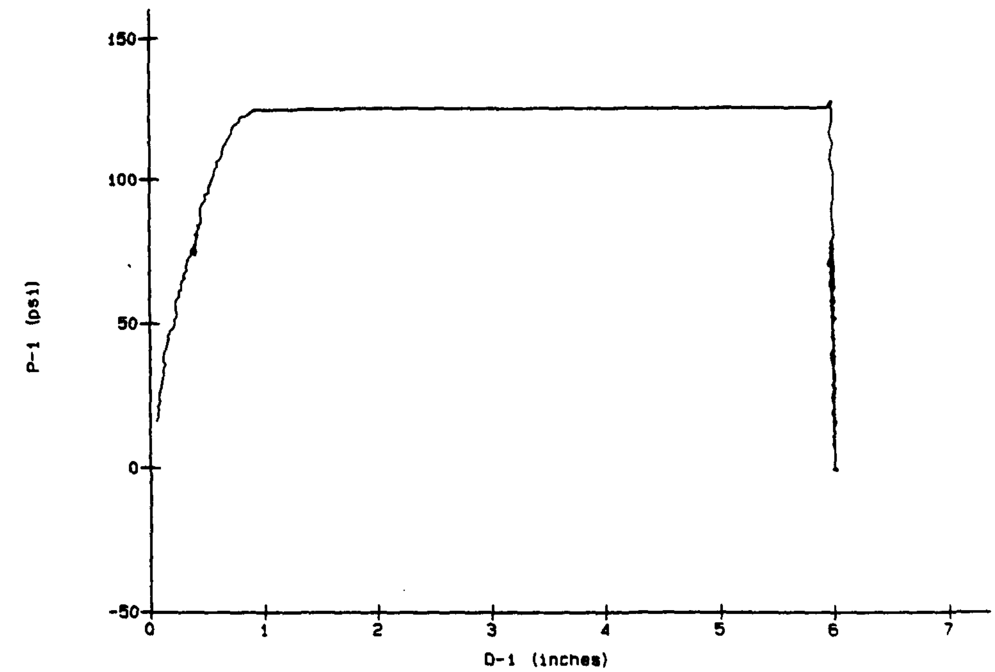

SLAB 15
150
100
50
0
-50
P-1 (psi)
0
50
100
150
200
250
300
350
400
SS-3 (micro inch/inch)


SLAB 15
150
100
50
0
-50
P-1 (psi)
-1500
-1000
-500
0
500
1000
1500
2000
SS-4 (micro inch/inch)


SLAB 15
P-1 (psi)
150
100
50
0
-50
0
200
400
600
800
1000
1200
1400
1600
1800
SS-5 (micro inch/inch)


SLAB 16
P-1 (psi)
150
100
50
0
-50
0
50
100
150
200
250
TIME (seconds)

SLAB 16



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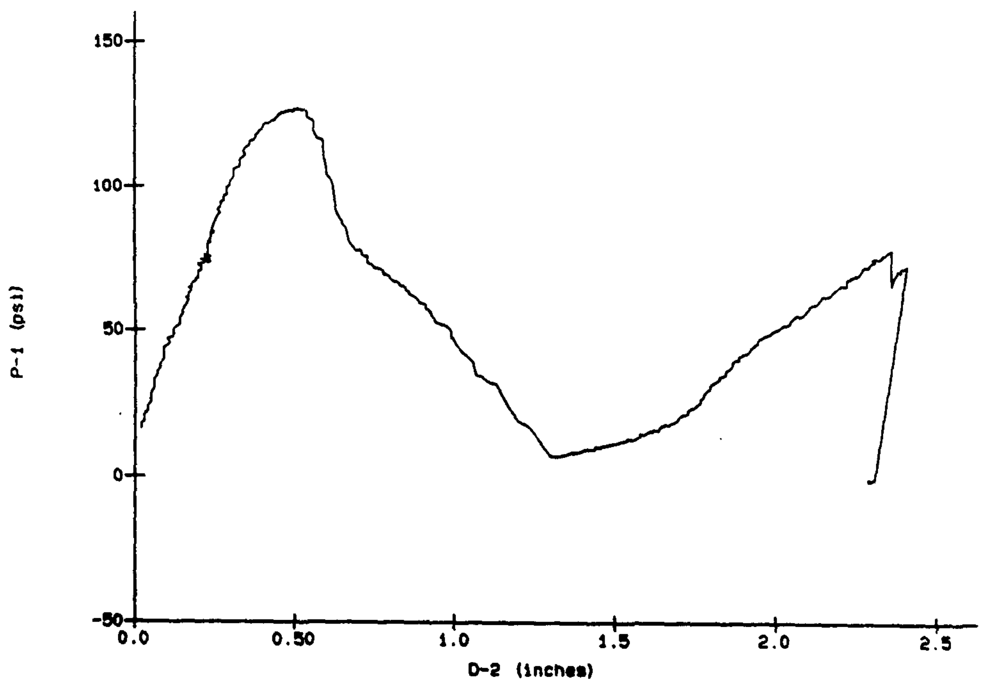

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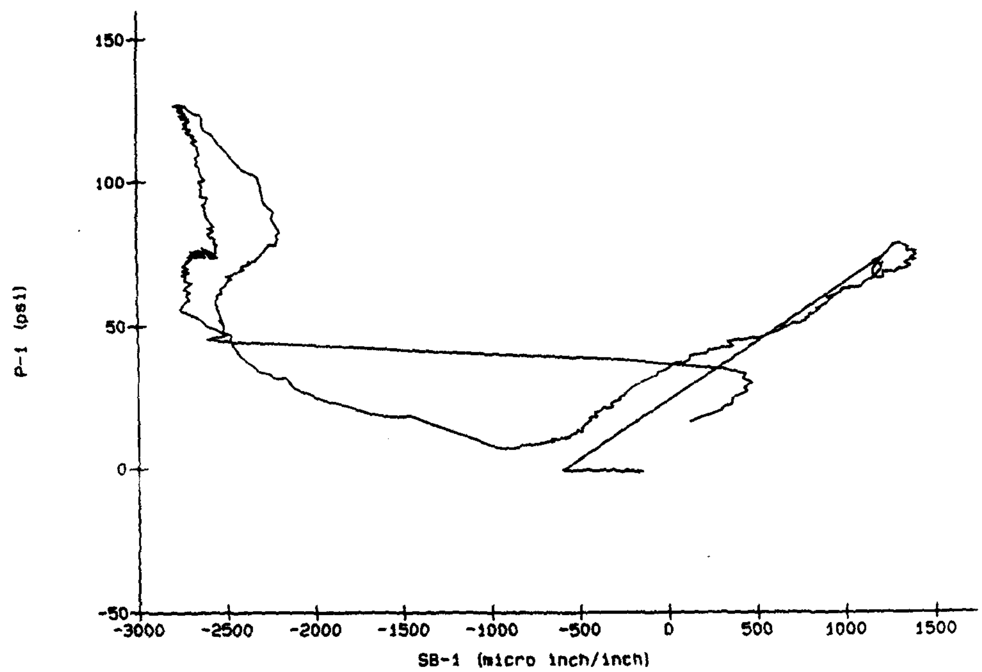

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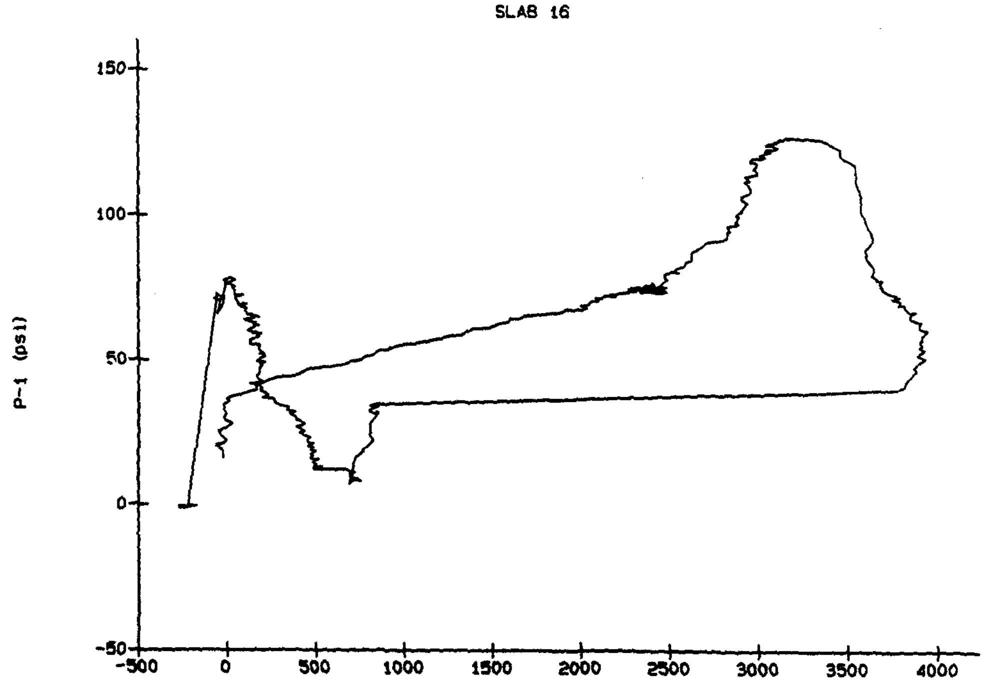
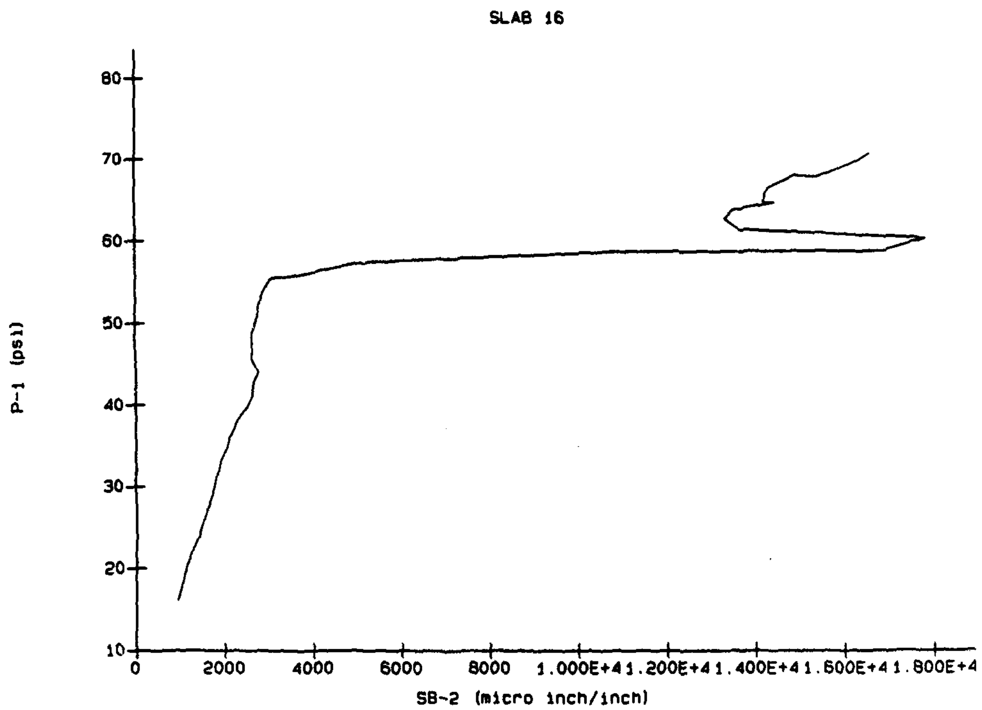
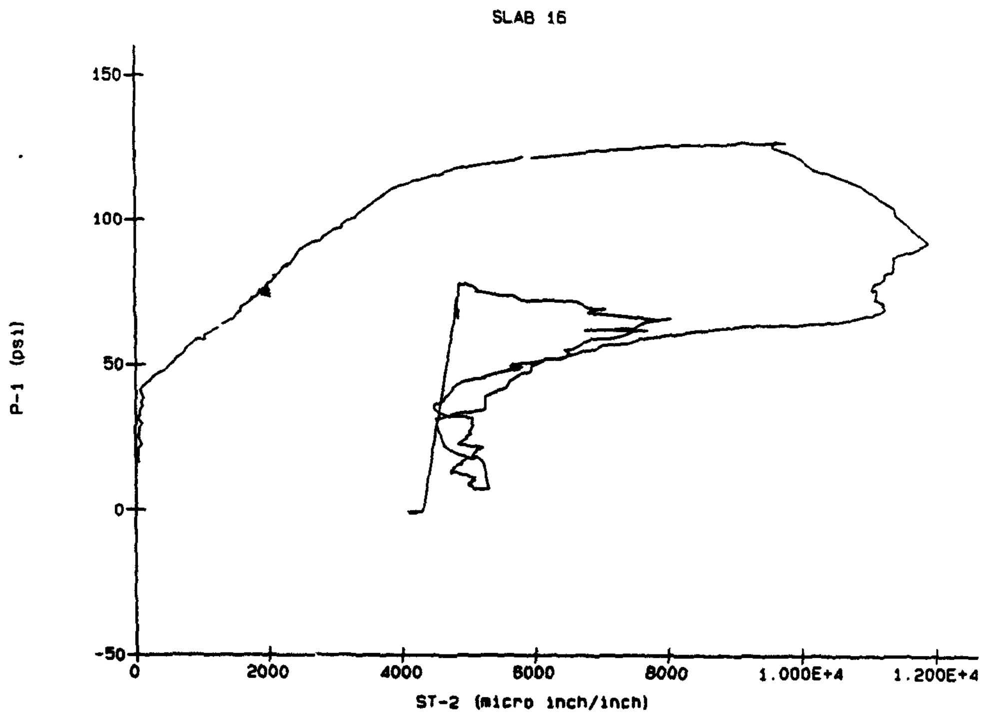
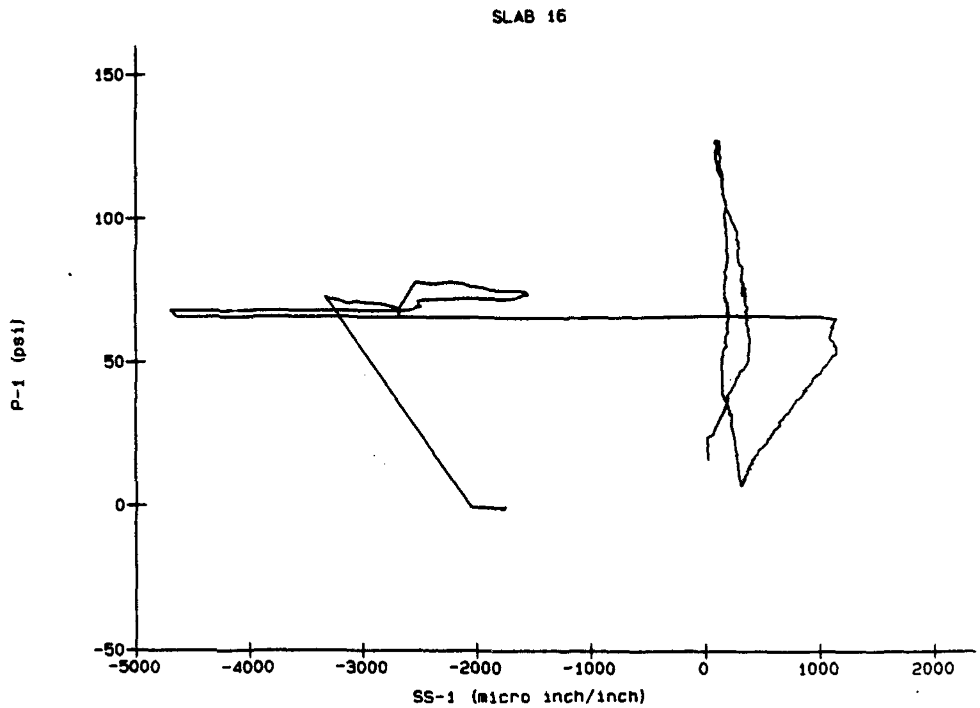
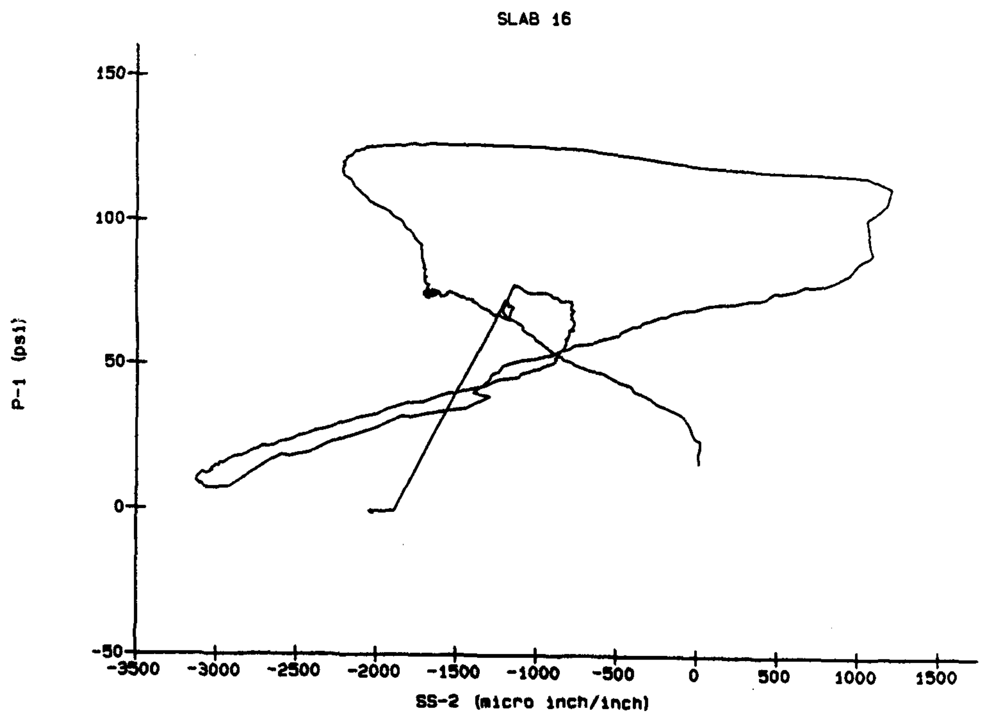
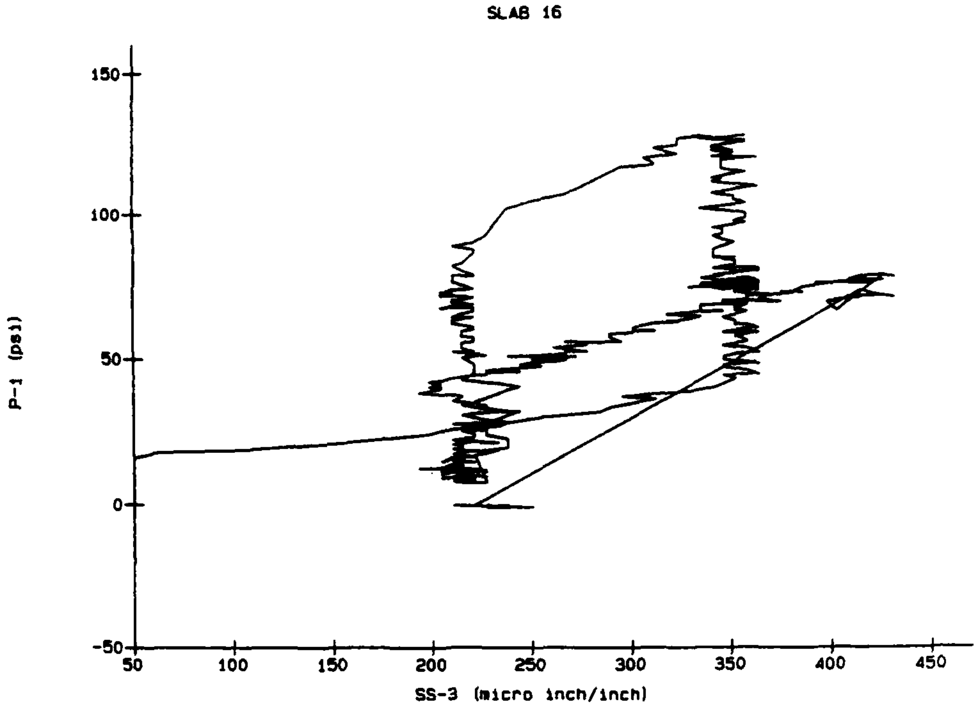


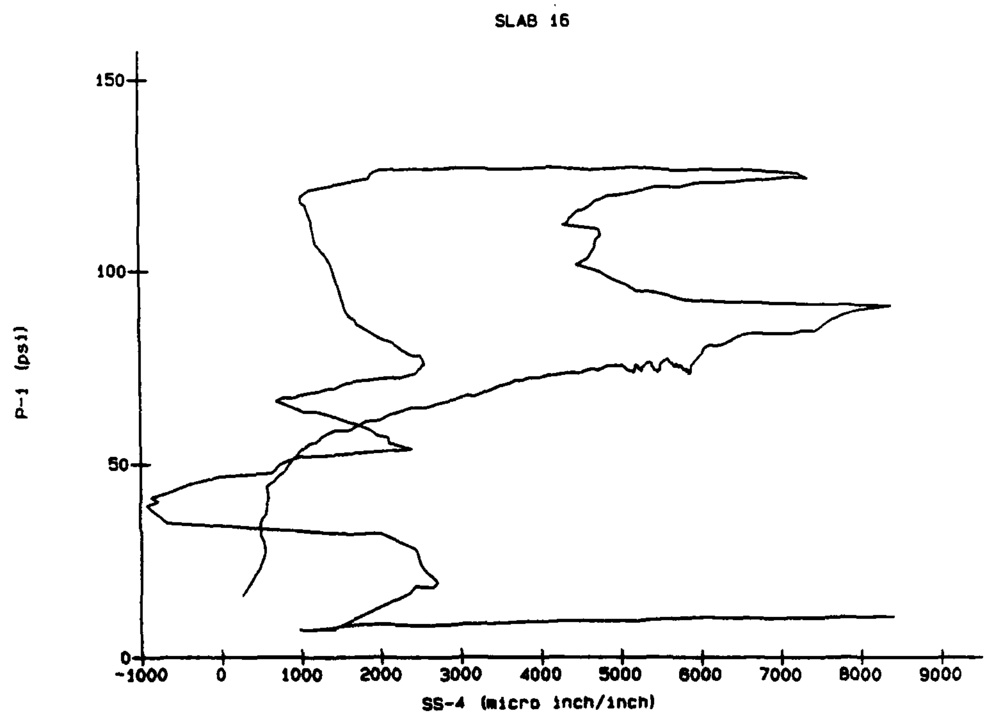

SLAB 16
80
70
60
50
40
30
20
10
0
2000
4000
6000
8000
1.000E+4
1.200E+4
1.400E+4
1.600E+4
1.800E+4
SB-2 (micro inch/inch)
P-1 (psi)

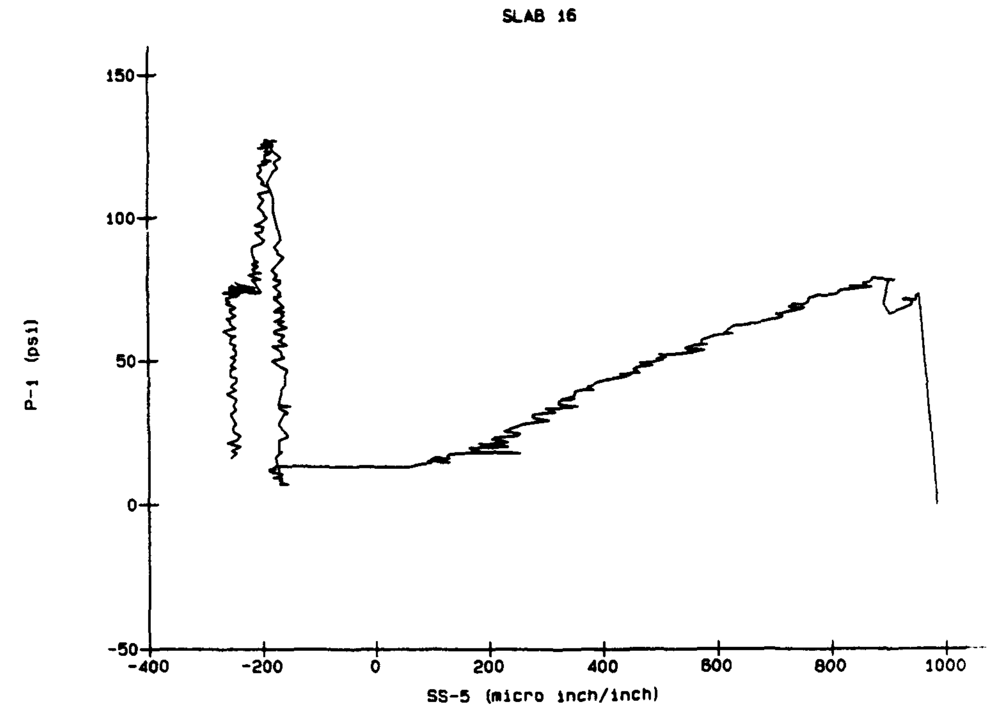

SLAB 16
150
100
50
0
-50
0
2000
4000
6000
8000
1.000E+4
1.200E+4
ST-2 (micro inch/inch)
P-1 (psi)


SLAB 16
150
100
50
0
-50
P-1 (psi)
-5000
-4000
-3000
-2000
-1000
0
1000
2000
SS-1 (micro inch/inch)


SLAB 16
150
100
50
0
-50
P-1 (psi)
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
SS-2 (micro inch/inch)


SLAB 16
P-1 (psi)
SS-3 (micro inch/inch)
150
100
50
0
-50
50
100
150
200
250
300
350
400
450


SLAB 16
P-1 (psi)
SS-4 (micro inch/inch)
150
100
50
0
-1000
0
1000
2000
3000
4000
5000
6000
7000
8000
9000


SLAB 16
150
100
50
0
-50
P-1 (psi)
-400
-200
0
200
400
600
800
1000
SS-5 (micro inch/inch)

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Stanley C. Woodson

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13. ABSTRACT *(Maximum 200 words)*

Design guides and manuals for blast-resistant reinforced concrete structures require the use of shear reinforcement (lacing bars or stirrups) to improve performance in the large-deflection region of response. It is generally known that the cost of using lacing reinforcement is considerably greater than that of using single-leg stirrups due to the more complicated fabrication and installation procedures. A thorough study of the role of shear reinforcement in structures designed to resist blast loadings or undergo large deflections has never been conducted. A better understanding of the effects of shear reinforcement on large deflection behavior will allow the designer to determine the benefits of using shear reinforcement and to determine which type is most desirable for the given structure. This capability will result in more efficient or effective designs as reflected by lower cost structures. The most comprehensive collection of past data relating design/construction details to slab response is included. Results of an experimental study comparing the effects of stirrups and lacing are presented, and recommendations regarding shear reinforcement design for blast-resistance structures are given.

14. SUBJECT TERMS
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